VOLUME 85 NO. WW2
JUNE 1959
PART 1

JOURNAL of the

Waterways and Harbors Division

PROCEEDINGS OF THE
AMERICAN SOCIETY
OF CIVIL ENGINEERS

BASIC REQUIREMENTS FOR MANUSCRIPTS

This Journal represents an effort by the Society to deliver information to the reader with the greatest possible speed. To this end the material herein has none of the usual editing required in more formal publications.

Original papers and discussions of current papers should be submitted to the Manager of Technical Publications, ASCE. Authors should indicate the technical division to which the paper should be referred. The final date on which a discussion should reach the Society is given as a footnote with each paper. Those who are planning to submit material will expedite the review and publication procedures by complying with the following basic requirements:

1. Titles should have a length not exceeding 50 characters and spaces.

2. A 50-word summary should accompany the paper.

3. The manuscript (a ribbon copy and two copies) should be double-spaced on one side of 8½-in. by 11-in. paper. Papers that were originally prepared for oral presentation must be rewritten into the third person before being submitted.

4. The author's full name, Society membership grade, and footnote reference stating present employment should appear on the first page of the paper.

5. Mathematics are reproduced directly from the copy that is submitted. Because of this, it is necessary that capital letters be drawn, in black ink, ⅛-in. high (with all other symbols and characters in the proportions dictated by standard drafting practice) and that no line of mathematics be longer than 6½-in. Ribbon copies of typed equations may be used but they will be proportionately smaller on the printed version.

6. Tables should be typed (ribbon copies) on one side of 8½-in. by 11-in. paper with a 6½-in. by 10½-in. invisible frame. Small tables should be grouped within this frame. Specific reference and explanation should be made in the text for each table.

7. Illustrations should be drawn in black ink on one side of 8½-in. by 11-in. paper within an invisible frame that measures 6½-in. by 10½-in.; the caption should also be included within the frame. Because illustrations will be reduced to 69% of the original size, the capital letters should be ⅛-in. high. Photographs should be submitted as glossy prints in a size that is less than 6½-in. by 10½-in. Explanations and descriptions should be made within the text for each illustration.

8. Papers should average about 12,000 words in length and should be no longer than 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

Further information concerning the preparation of technical papers is contained in the "Technical Publications Handbook" which can be obtained from the Society.

This Journal is published quarterly by the American Society of Civil Engineers. Publication office is at 2500 South State Street, Ann Arbor, Michigan. Editorial and General Offices are at 33 West 39 Street, New York 18, New York. $4.00 of a member's dues are applied as a subscription to this Journal. Second-class postage paid at Ann Arbor, Michigan.

EM,IR,WW.

Journal of the

WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

June, 1959

Papers

20

li
co
w
du
sa
it

C
P
ge
m
b
d
la

bl
c
th
h
r
a
B
a

N

1

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

SAND BYPASSING AT SANTA BARBARA, CALIFORNIA

R. L. Wiegel[1]

ABSTRACT

A breakwater constructed at Santa Barbara, California, interrupted the littoral drift of sand. The man-made harbor filled with sand, and the downcoast beaches were nearly completely stripped of sand. At first the harbor was dredged periodically, and lately nearly continuously, with the sand being dumped downcoast to maintain these beaches. Although costly, and not always satisfactory in all details, this method has in general adequately performed its function.

INTRODUCTION

Santa Barbara is located on a sandy lowland on the southern coast of California in an area afforded considerable protection from waves of the Pacific Ocean by offshore islands (Fig. 1). The coastal region in this area is generally rugged with mountains extending almost to the shore. There are many rock headlands separated by coves with narrow cobble, gravel, or sand beaches. There are no large rivers, but there are many steep streams which drain narrow basins averaging 5 miles in width which when in flood carry large quantities of sediments and debris.(16)

The major source of sand for the beaches in the Santa Barbara area probably comes from the streams feeding into the ocean.(4,13,14,16,18) Some sand comes from the erosion of the coastal cliffs, but it is a common observation that this erosion has been small in this area in recent historical times. It has been determined(13,14) that some sand does move into this area from the region upcoast from Pt. Conception and Pt. Arguello. Other possible sources are sand blown by the wind and sand from offshore areas. Observations by Bascom(1) during times of high winds indicate that sand blown by the wind is a relatively unimportant source of sand for the beaches. Because of the

Note: Discussion open until November 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2066 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. WW 2, June, 1959.

1. Associate Research Engr., Univ. of Calif., Berkeley, Calif.

FIGURE 1

relative importance of the streams as a source of sand for this region, the amount of sand should generally be strongly dependent upon the amount of rainfall in the area. In particular it should be dependent upon the number of storms that produce high rates of runoff, as it is this condition that results in a large amount of sand being carried to the littoral region. A good correlation between littoral drift of sand and rainfall in this area has been shown,[16, 18] although it must be cautioned that other effects of the same storms, such as the intensity and direction of the waves resulting from the storms, must be considered. From the long range viewpoint the program of flood control and soil erosion prevention within the United States will probably seriously affect the amount of sediment moving to the ocean, and make more serious the problem of coastal erosion.

The median diameter of the sand on the beach face and the slope of the beach face are best presented in relationship to other beaches on the Pacific Coast of the U. S.,[2] as shown in Fig. 2.

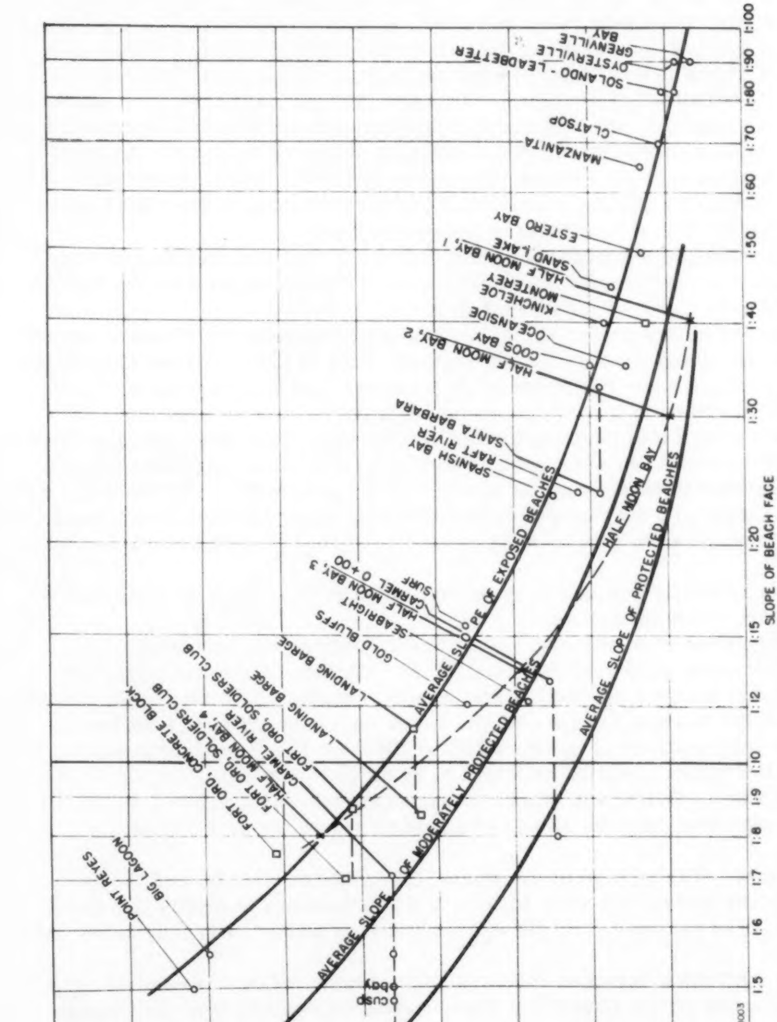
The waves in this region are either generated between the Channel Islands and the coast (local wind waves) or are generated in the ocean seaward of the islands, perhaps many thousands of miles away. Local observations[16,18] lead to the conclusion that the predominant waves are from a westerly direction entering Santa Barbara Channel between Pt. Conception and San Miguel Island. This results in a wave-generated longshore drift from west to east. Current measurements along the shoreline[16] confirm this. Waves seldom exceed a height of 3 feet except during times of storms from the southeast,[16] with the average wave period being about 12 sec, but ranging from 8 sec to 16 sec.[6]

The net result of this is a gradual drift of sand from west to east, except during short intervals of time.

The refraction of waves arriving at Santa Barbara has been studied by O'Brien[10] using graphical methods.[5] He concluded from the refraction drawings that waves from the general westerly direction would almost always break at about the same angle with the beach (an observed fact) even though they were originally from the W, WSW, or SW (see Fig. 3 for an example). In addition, the waves would be less high at Santa Barbara than along the coast west of the city. Refraction diagrams drawn for waves from the S and SE directions show that only the SE waves would be relatively high at Santa Barbara (Fig. 3).

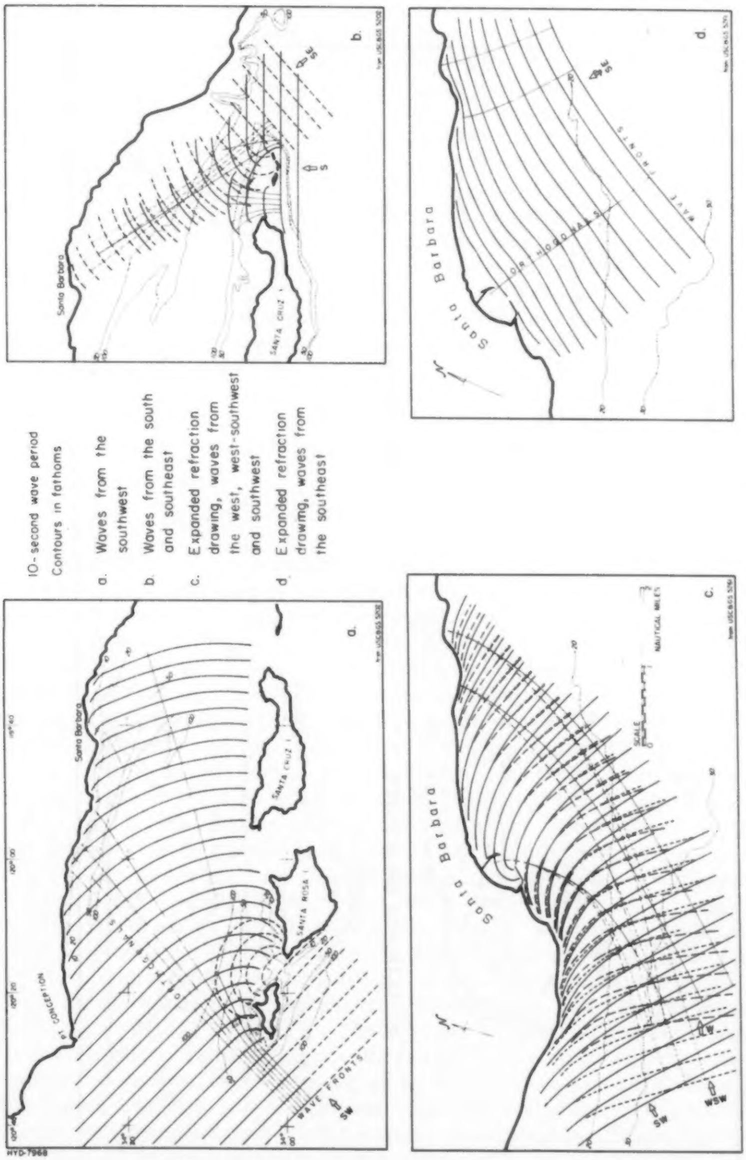
As a result of the origin of the waves, the offshore islands, and the refraction of the waves, the wave pattern at Santa Barbara is nearly the same for all times of the year, with the exception of the waves from the occasional SE storms.

During 1927-28 a detached rubble-mound breakwater with a concrete cap was constructed off Pt. Castillo at Santa Barbara (Fig. 4A).[16] The breakwater was about 1,425 ft. in length roughly parallel to shore, with an arm heading towards shore which was about 400 ft. long. Because waves refract in shoaling water as well as diffract in the lee of a breakwater, the waves from the west moved in an easterly direction through the gap between the shore and the breakwater, while the waves in the vicinity of the easterly tip of the breakwater swung around in such a manner that they moved in a westerly direction in the gap area. This interaction of the waves caused sand to deposit in this area, causing the harbor to shoal. Because of this shoaling, in 1930[16] the breakwater was extended to shore by an additional 600 ft. of structure (Fig. 4b).



SIZE-SLOPE RELATIONSHIP AT THE REFERENCE POINT
(after Bascom, 1951)

FIGURE 2



Refraction Diagrams for Santa Barbara and Vicinity for Waves of 10-second Period

(after O'Brien, 1950)

FIGURE 3

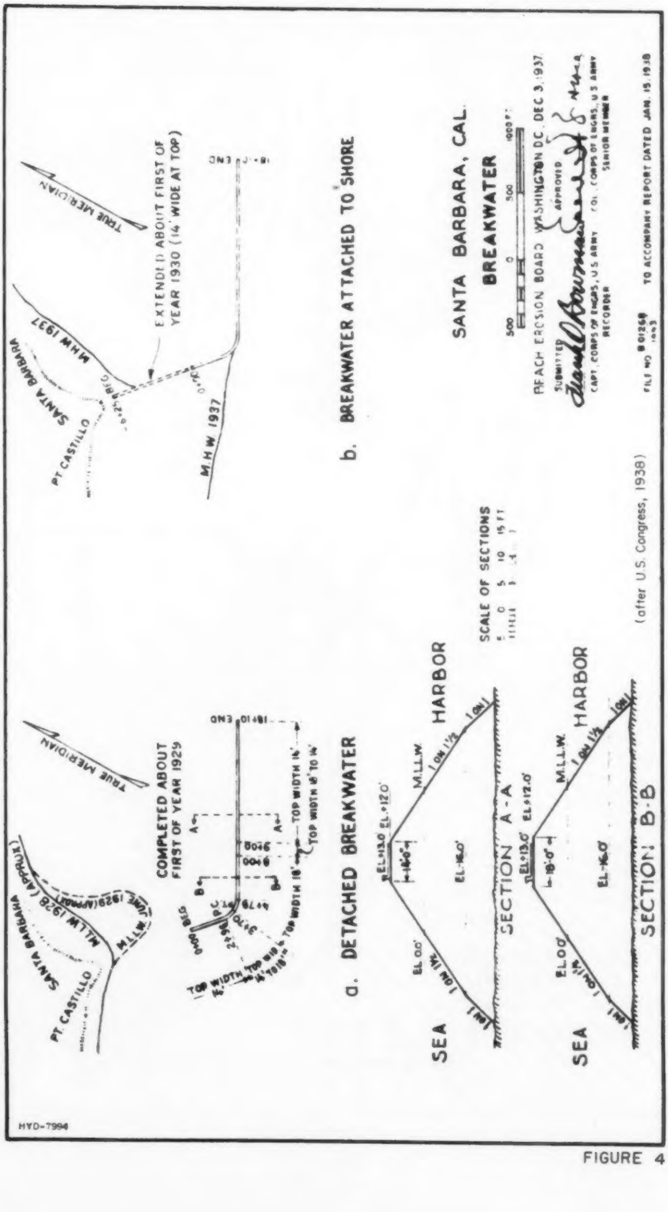

a. DETACHED BREAKWATER
b. BREAKWATER ATTACHED TO SHORE
SANTA BARBARA, CAL.
BREAKWATER
SECTION A-A
SECTION B-B
SEA
HARBOR
TRUE MERIDIAN
PT. CASTILLO
SANTA BARBARA
COMPLETED ABOUT FIRST OF YEAR 1929
EXTENDED ABOUT FIRST OF YEAR 1930 (14' WIDE AT TOP)
M.H.W. 1937
SCALE OF SECTIONS
BEACH EROSION BOARD WASHINGTON D.C. DEC 3, 1937
TO ACCOMPANY REPORT DATED JAN. 15, 1938
(after U.S. Congress, 1938)
HYD-7994

FIGURE 4

This breakwater proved to be an effective trap of the sand that was moving in the littoral stream. This sand filled the area west of the breakwater, moving along the breakwater, swinging around its tip, and eventually moving into the harbor (Fig. 5).[8] Thus, one end result was the filling of the harbor with sand (Fig. 6). From measurements made of the rate of fill west of the breakwater an annual drift of about 270,000 cubic yards (about 740 cubic yards per day) was computed.[16] The waves acting on the beach immediately east of the breakwater continued to move sand eastward; however, this sand was not replaced because the sand that would have been normally deposited on it was trapped by the breakwater. This problem gradually worked its way eastward. Thus, a second end result was the erosion of the beaches to the east of the breakwater. An example of this erosion has been shown in Fig. 7. The amount of sand lost was actually worse than this figure indicates, as the 1930 survey was made in February when the beaches in this area are normally denuded due to sand moving offshore, while the 1932 survey was made in August when the beaches are built seaward by the sand moving onshore.[7,12,20]

Engineering studies were made of the problems and a decision was reached to dredge the harbor and to dump the sand east of the harbor in such a manner that this sand would be moved by the waves along the coast to build up the beaches and to maintain them in a desirable condition.

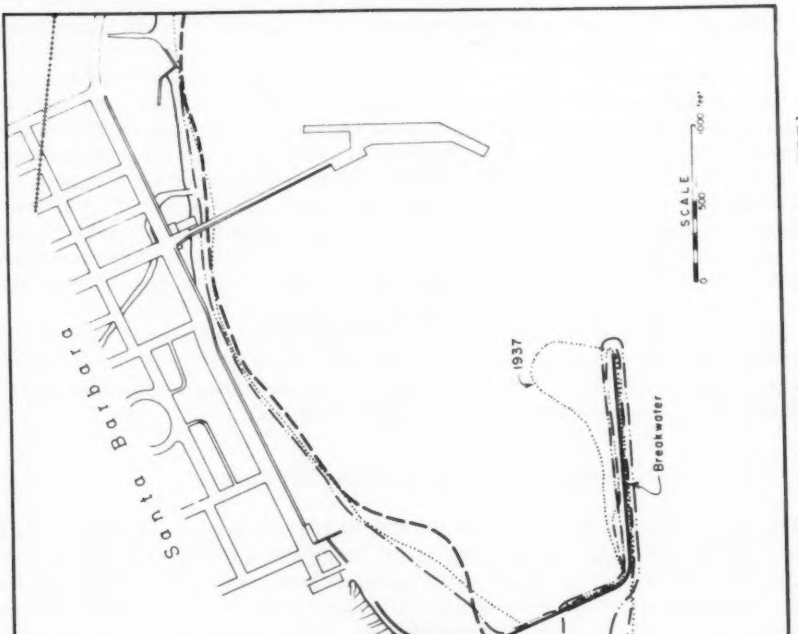
Periodic Dredging and Sand Bypassing

The harbor was first dredged in the fall of 1935 by hopper dredges, with a little over 200,000 cubic yards of material being moved.[16] Material was dumped in about 22 feet of water approximately one mile east of the breakwater and about 1,000 ft. from shore. It formed a mound about 2,200 feet long and 5 feet high. It was expected that the waves would move the sand onshore and eastward. However, this did not occur as is evident in the chart of the May 1937 survey (Fig. 8). Surveys made in 1946[18] showed that the mound at that time was at no point more than a foot below its 1937 depth.

As a result of this loss of material insofar as beach maintenance was concerned it was decided to pump the material by pipeline in future dredgings and deposit on the "feeder beach" just east of the outfall sewer line (Fig. 7). The disposal areas for the 1938, 1940, 1942 and 1945 dredgings are shown in Fig. 9. Surveys of the effect of the sand moving from the feeder beach eastward showed that the shorter, wider feeder beaches of the 1940 and 1942 dredgings were about as effective as the longer, narrower feeder beach of 1938, but that the shortest, widest feeder beach (1945) was not so effective.[18]

For complete details of the effectiveness of this method of sand bypassing, References 16 and 18 should be consulted. However, a representative example is shown in Fig. 9, 10 and 11. The net loss of material from the western side of East Beach is because this is the feeder beach. It is evident that waves moved the sand eastward causing an increase in the amount of sand all along the beach, except in a few locations. The determination of the reasons for the regions of opposite trend should require considerable attention in the future.

It should be emphasized that waves deposited the sand not only on the beach but for an appreciable distance offshore. Hence a large portion of the sand necessary to replenish the beach is not noticed by the layman or the casual observer.



Shoreline Changes Upcoast of the Santa Barbara Breakwater (after Johnson, 1957)

FIGURE 5

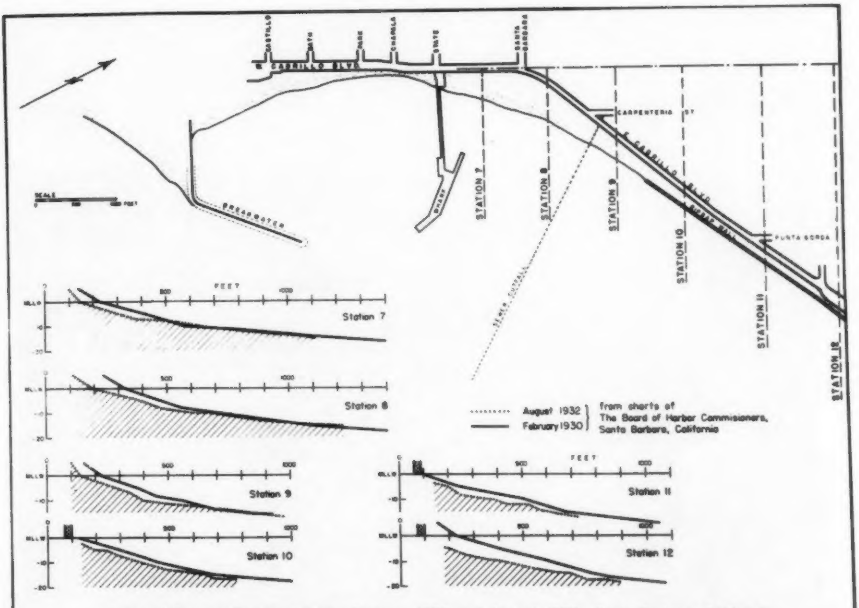
HYD-7995

Figure 6. Aerial Photograph of Santa Barbara Harbor Looking Upcoast, 31 January 1947

A summary of the dredging operations are shown in Table I. With the exception of the 1935 dredging data, the data presented in Table I were obtained from the U. S. Army Engineer District, Los Angeles. For historical reasons an aerial photograph of the dredging operation of 1938 is shown in Fig. 12.

One of the most interesting and certainly one of the most important phenomena associated with the sand bypassing problem is the formation of the sand deposit at the end of the breakwater. The harbor in this area was dredged every two years or so, a portion of it to about 30 feet below MLLW (mean lower low water) and a portion to about 20 feet below MLLW. An example of the bottom profiles just after a dredging are shown in Fig. 13, together with the bottom profiles approximately 2-1/2 years after the dredging.

The build up of this sand spit is remarkably consistent. In Fig. 14 are shown the MLLW profiles for seven cases. There are two modes, one



Loss of Sand on Beaches between February 1930 and August 1932, Santa Barbara, California

FIGURE 7

perpendicular to the breakwater and one making an angle of about 60 degrees with the breakwater. Why there should be two modes is not known, but it may be associated with the effects of the occasional SE storms, or with the strength of the littoral current, the latter possibility being indicated by a model study described later in this paper. The change of the size and shape of the MLLW contour with time between two sets of dredgings is shown in Fig. 15. It is not known why the change in orientation towards the end of the buildup should have occurred. Some detailed cross sections of the development of this sand spit are shown in Fig. 16.

One remarkable characteristic of the spit is the abrupt change in slope, called the breakoff line (Fig. 17). The relationship between the area defined by the breakoff and the volume of this fill is remarkably consistent as can be seen in Fig. 18.[1,15] In Figs. 19 and 20 are shown the rates of fill for the interval for which Fig. 18 was constructed.

The amount of material accumulated within the harbor (primarily the sand spit for the years 1932 through 1951 is shown in Fig. 21. These data are also tabulated in Table II.

In 1937 a model study was made by Lapsley[9] of the effect of the breakwater on the sand movement at Santa Barbara. This study was made with a model of 1:750 horizontal scale and 1:100 vertical scale. Clean quartz sand with a median diameter of 0.35 mm. and specific gravity of 2.62 was used. The wave steepness used during the tests corresponded to a prototype condition of continuous local storm waves. In spite of these discrepancies between prototype and model conditions, the results of the tests were informative. It was found that sand could be transported at the rate of 2.6 cu. in. per minute in the model. Taking the average rate of sand transport in the

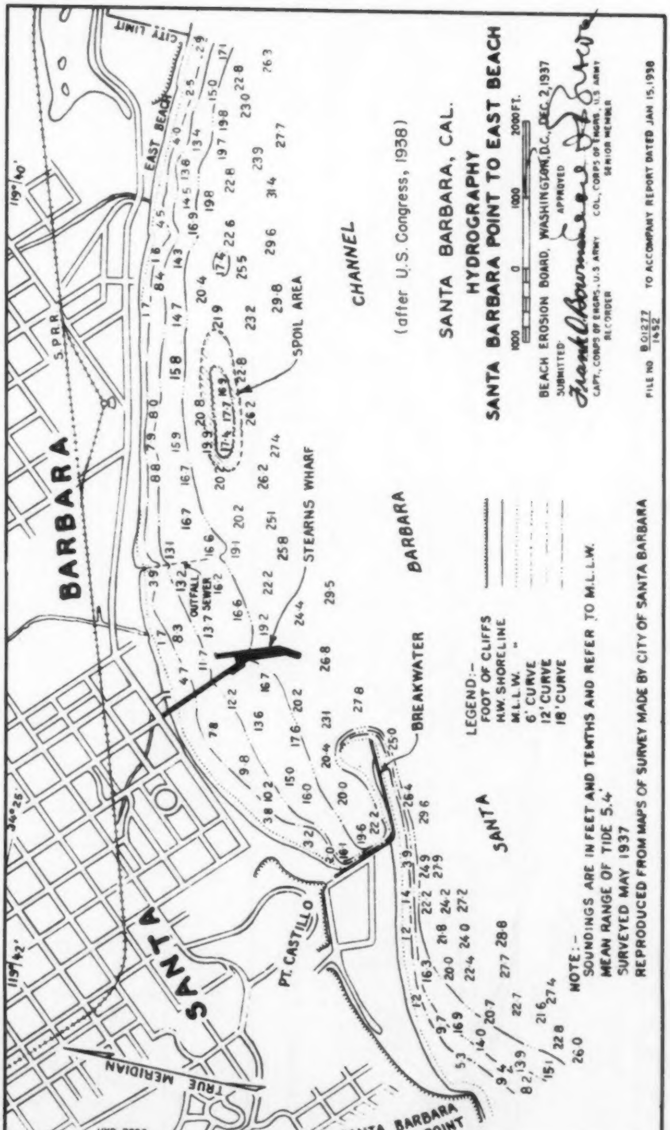

SANTA BARBARA, CAL.
HYDROGRAPHY
SANTA BARBARA POINT TO EAST BEACH
(after U.S. Congress, 1938)
BEACH EROSION BOARD, WASHINGTON, D.C., DEC. 2, 1937
SUBMITTED
APPROVED
CAPT., CORPS OF ENGRS., U.S. ARMY RECORDER
COL., CORPS OF ENGRS., U.S. ARMY SENIOR MEMBER
FILE NO B01277 1452
TO ACCOMPANY REPORT DATED JAN 15, 1938
LEGEND:-
FOOT OF CLIFFS
H.W. SHORELINE
M.L.L.W.
6' CURVE
12' CURVE
18' CURVE
NOTE:-
SOUNDINGS ARE IN FEET AND TENTHS AND REFER TO M.L.L.W.
MEAN RANGE OF TIDE 5.4'
SURVEYED MAY 1937
REPRODUCED FROM MAPS OF SURVEY MADE BY CITY OF SANTA BARBARA
SANTA BARBARA
CHANNEL
SANTA BARBARA
SPOIL AREA
STEARNS WHARF
BREAKWATER
OUTFALL SEWER
EAST BEACH
CITY LIMIT
S.P.R.R.
PT. CASTILLO
SANTA BARBARA POINT
TRUE MERIDIAN
HYD-7996

FIGURE 8


GAINS OR LOSSES OF BEACH MATERIAL AS DETERMINED FROM SEMIANNUAL SURVEYS
SECTION 1 - EAST BEACH TO EAST CITY BOUNDARY
SECTION 2 - EAST CITY BOUNDARY TO MIRAMAR
SECTION 3 - MIRAMAR TO ORTEGA HILL
GAINS OR LOSSES OF BEACH MATERIAL BETWEEN SEMIANNUAL SURVEYS
MEAN HIGHER HIGH WATER SHORELINE CHANGES
RECORD OF FEEDER BEACH OPERATION
LEGEND
East Beach
Summerland
(after U.S. Congress, 1948)

FIGURE 9

TABLE I

SANTA BARBARA HARBOR
MAINTENANCE DREDGING

Year (a)	Contract Unit Price (b)	Pay Quantity (c)	Contract Payment (d)	Govt. Cost (e)	Non-Pay Yardage (f)	Gross Yardage (g)	Total Cost (h)	Cost Per Yard (i)
			(b x c)			(c + f)	(d + e)	(h)/g
Oct-Nov. 1952	0.2647	1,070,000	$ 283,670	$17,660	104,000	1,174,000	$ 301,330	0.2567
May-June 1949	0.1838	704,106	$129,414.68	$18,624.91)	82,419	838,152	$160,703.59	0.1917
Navy (Part of Area "C")	0.22	51,627	$ 11,357.94	$ 1,306.06)				
May-June 1947	0.1739	675,044	$100,000.15	$ 9,999.85	68,933	642,977	$110,000.00	0.1711
June-Sept 1945	0.2204*	717,773	$158,171.16	$11,828.84	0	717,773	$170,000.00	0.2368
July-Aug 1942	0.218	558,610**	$121,776.98	$ 9,531.97	41,500	600,110**	$131,308.95	0.2188
Inside breakwater west of Sec. B	-	38,690	2,400	Paid for by local interests $100/hr. for 24 hrs.				0.6203
June-July 1940	0.178	646,067	$114,999.93	$ 9,532.05	51,652	697,719	$124,531.98	0.1785
May-June 1933	.21	582,000	$122,220.00	$ 5,884.11	24,427	606.427	$128,104.11	0.2112

* Bid price 24.5¢ based on 420,500
Additional 300,000 at 18.5¢ by supplemental agreement

** Includes about 103,000 cubic yards dredged for the Navy in Area "C" (vicinity of Coast Guard pier and approach)

The above data were obtained from the U. S. Army Engineer District, Los Angeles, Ltr. SPLGP-R, 28 Oct. 1958.

Sept. 1935						202,000	$ 30,000	0.1485

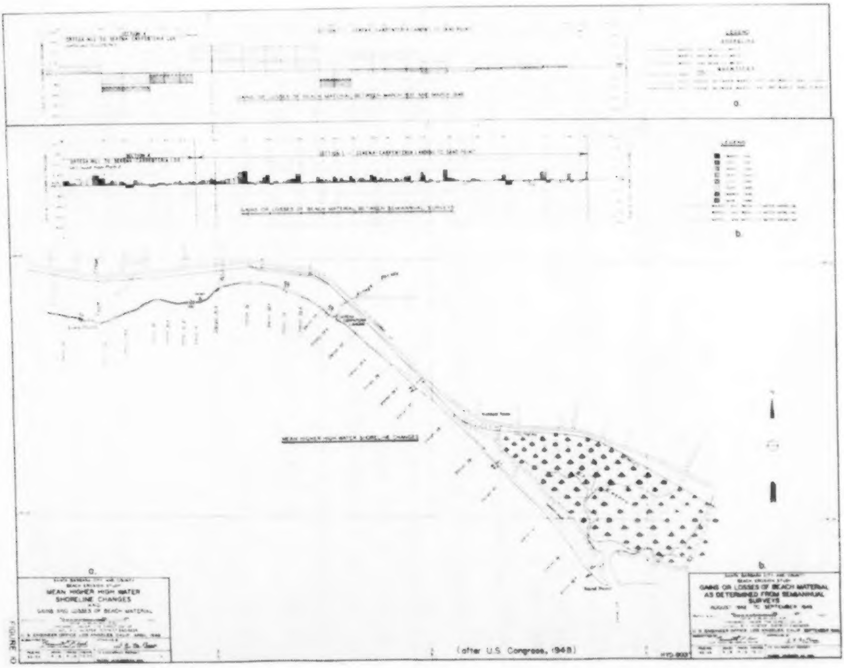


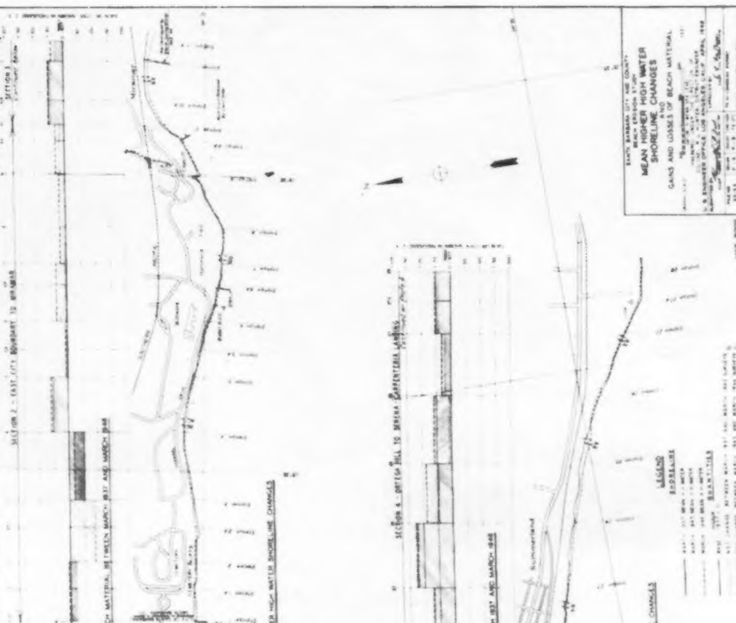
FIGURE 10

prototype as 740 cu. yds. per day results in a time scale such that one minute in the model corresponds to 4.2 days in the prototype.

Some results of the study are shown in the series of photographs reproduced in Fig. 22. Photographs (a) through (d) show the time history of buildup of the sand in the vicinity of the breakwater. The sand at the base of the breakwater within the harbor is from a prior study made of just the detached breakwater and does not affect the later study. It was found that after the sand had moved to the end of the breakwater the current parallel to the breakwater was so strong that it swept the sand towards stearns wharf (Fig. 22, c. d). The model breakwater was constructed of a strip of heavy sheet metal, impervious and vertical. After the photograph in Fig. 22d was taken small rocks were cemented to the breakwater to decrease the current strength. It was then found that the sand swung around as shown in Fig. 22c. Another complete test was made using the new breakwater with the results as shown in Fig. 22f.

Lapsley stated that he was never able to reproduce in the model the sand condition inside the breakwater that extended from the spit to the base of the breakwater arm. It has recently been found that this condition resulted from the movement of sand through the "porous" breakwater, and this movement has now been prevented by grouting the breakwater.

Although the model did reproduce certain characteristics of the sand movement, it did not predict the amount of sand that would be moved, the width of the beach, etc.



(after U S Congress, 1948)

FIGURE 11

Aerial Photograph of Dredging Operations, 21 June 1938, Santa Barbara, California (Spence Air-Photos)

FIGURE 12

Continuous Dredging and Sand Bypassing

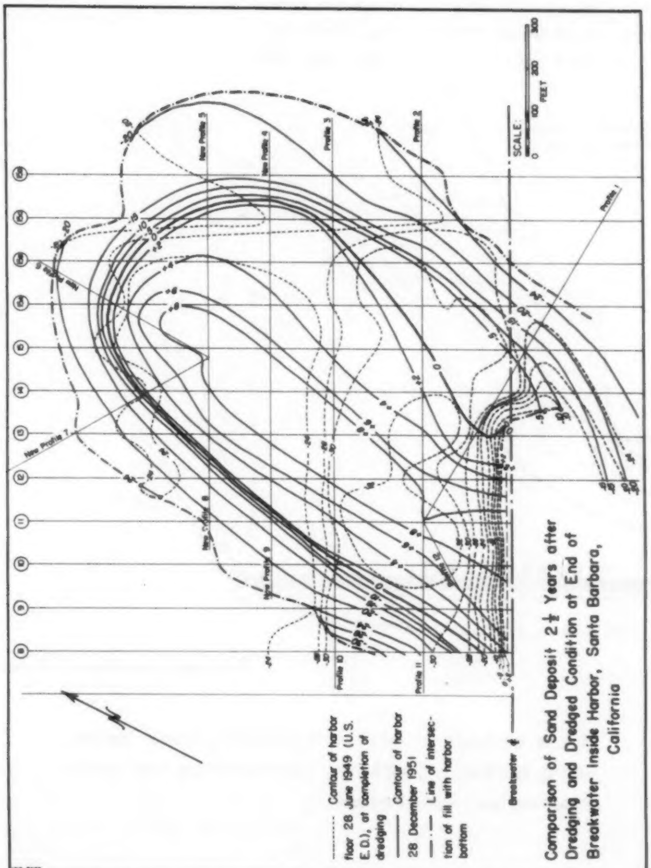
The last large dredging of the harbor was done during November and December 1952. The City of Santa Barbara decided to use the sand spit inside the end of the breakwater as a protection to the inner harbor from waves from a southerly direction, and to maintain an entrance into the inner harbor

Table II

Estimated Annual Volumes of Accretion in Santa Barbara Harbor, 1932-1951, Inclusive (from J. W. Johnson[6]).

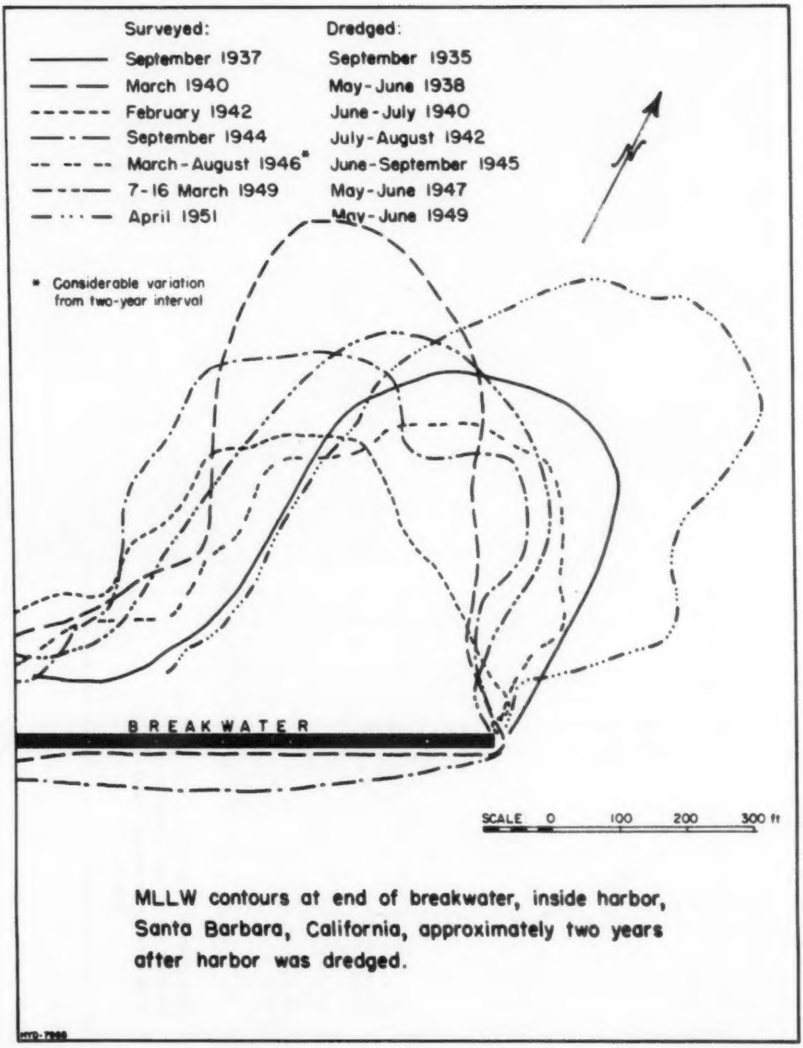
Year	Accretion cubic yards	Year	Accretion cubic yards
1932	225,000	1942	245,000
1933	265,000	1943	210,000
1934	390,000	1944	235,000
1934	200,000	1945	295,000
1936	225,000	1946	400,000
1937	205,000	1947	330,000
1938	235,000	1948	370,000
1939	280,000	1949	330,000
1940	310,000	1950	300,000
1941	260,000	1951	283,000

Average, 1932-51, incl. = 279,650 cu. yds. per year



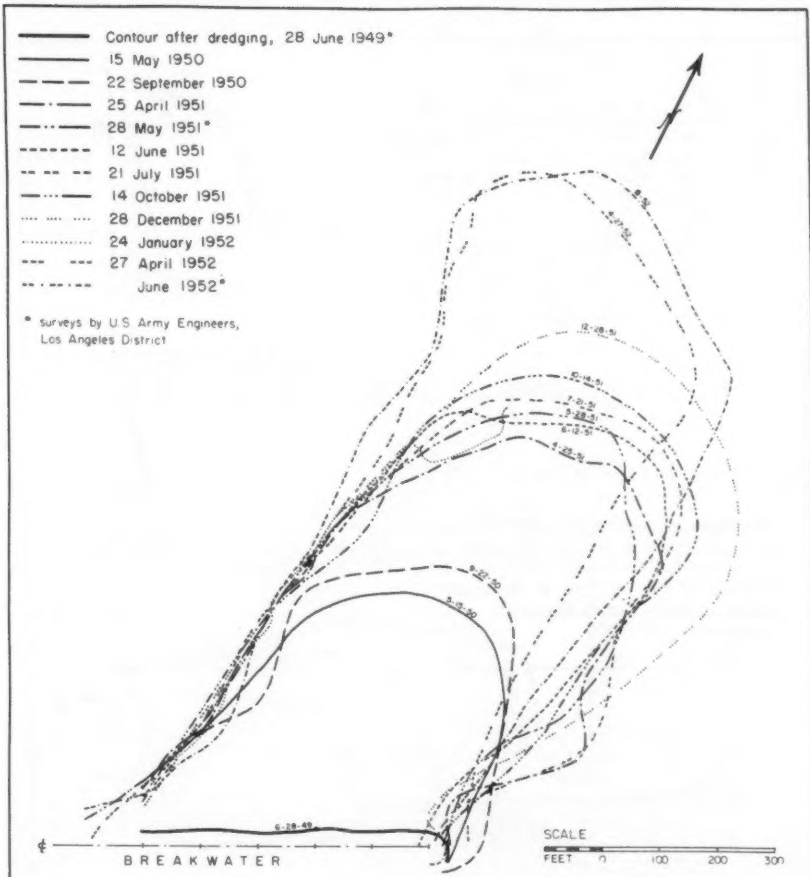
Comparison of Sand Deposit 2½ Years after Dredging and Dredged Condition at End of Breakwater Inside Harbor, Santa Barbara, California

FIGURE 13



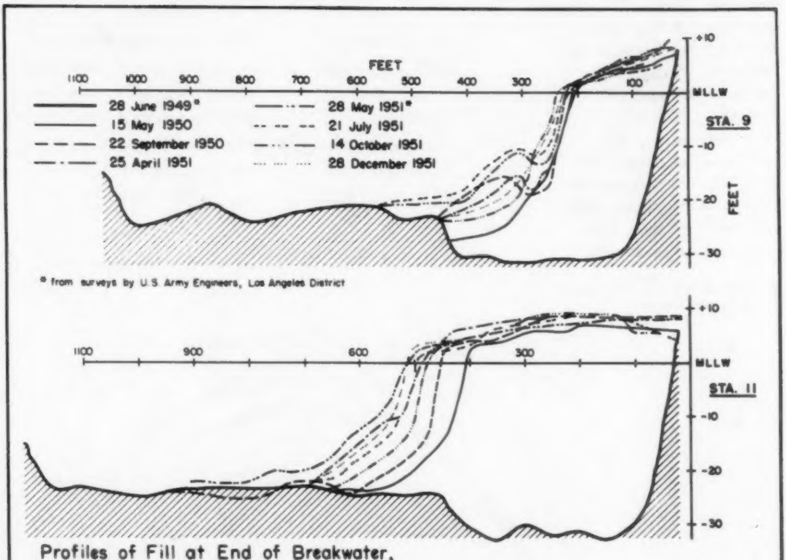
MLLW contours at end of breakwater, inside harbor, Santa Barbara, California, approximately two years after harbor was dredged.

FIGURE 14



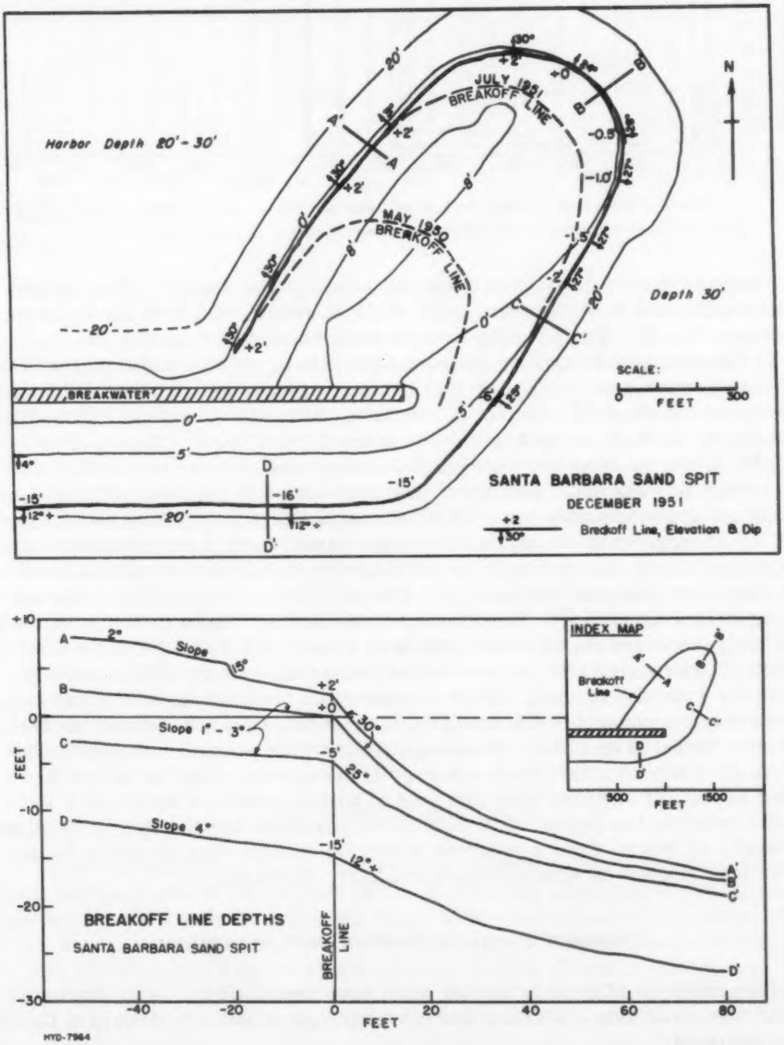
MLLW Contours at End of Breakwater, Inside Harbor, Santa Barbara, California, at Various Times after Harbor Was Dredged (May-June 1949)

FIGURE 15



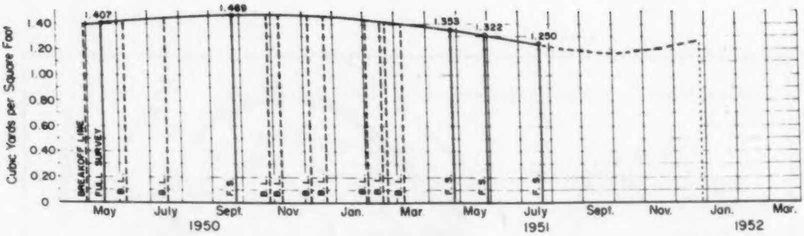
Profiles of Fill at End of Breakwater, Inside Harbor, Santa Barbara, California

FIGURE 16



General Characteristics of Fill at End of Breakwater, Inside Harbor, Santa Barbara, California (after Trask, 1952)

FIGURE 17



Volume Factor from 28 May 1950 to 28 December 1951. Volume factor = $\frac{\text{cu. yds. sand}}{\text{sq. ft. area breakoff line}}$, volume = area breakoff line x volume factor.

FIGURE 18

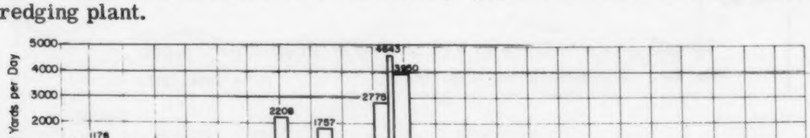
by means of nearly continuous dredging with a small dredge. The channel was to be maintained to a minimum depth of 15 ft. below MLLW in the location shown in Fig. 23. Dredging operations were started in August 1956.

The dredge had a capacity of about 1600 cubic yards of material per 8 hour shift, and experience has shown that because of wave and weather conditions it pumps only about 72 per cent of the time. Although this capacity is adequate on a yearly basis it is not adequate on a short term basis. For example, in Fig. 19, it can be seen that between the surveys of 15 Feb. and 20 Feb. 1951 an average of 4643 cubic yards per day were added to the sand spit, with most of this material probably being added during a SE storm lasting about one day.

This short term difficulty is even more pronounced if one considers that large amounts of material may be shifted a few hundred feet without even considering a net increase of material. For example, one SE storm occurred during Feb. 4-5, 1958.(3) The Public Works Dept. of Santa Barbara made spot checks of the water depth in the area both shortly before and shortly after the storm. It was stated that the storm had caused an unbelievable change to occur in the entrance channel. Many thousands of cubic yards of sand were moved into the channel. The westerly bank of the entrance channel shifted easterly from 100 to 400 ft. This nearly closed the entrance channel. The bottom contours obtained from a survey made shortly after the storm are shown in Fig. 23 together with two sets of bottom contours surveyed a few months prior to the storm. It should be emphasized that the spit is developed primarily by waves from a westerly direction, and as such as probably unstable with respect to waves from a southerly direction.

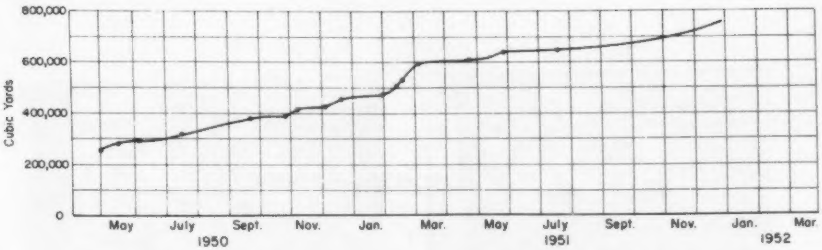
Proposed Future Methods of Sand Bypassing

Other methods of sand bypassing have been proposed for Santa Barbara. One of these methods utilizes a fixed dredging plant and one utilizes a floating dredging plant.



Rates of Harbor Accretion from 28 June 1949 to 28 December 1951.

FIGURE 19



Cumulative Volume of Harbor Accretion from 28 April 1950 to 28 December 1951.

FIGURE 20

The proposed fixed dredging plant has been described in Reference 17. The general layout is shown in Fig. 24. The initial cost of the installation was estimated to be $106,000 (in 1941) and the annual operating cost was estimated to be $67,700 (in 1941), based upon transporting 300,000 cubic yards of sand per year (22.6 cents per cubic yard). This system was conceived to operate with the existing breakwater, and it was expected that it would prevent the sand from moving around the breakwater by trapping the sand in the location shown in the plan drawing. One drawback to the system was the long distance through which the sand had to be pumped (from 5,000 to 6,000 feet, 16-in. dia., carrying about 10% solids by volume at 13 ft/sec.) with the high pumping costs associated with this distance (about a 1,000 horsepower pump).

The floating dredging plant system was conceived to work in conjunction with a modified harbor as shown in Fig. 25. In this plan a second breakwater would be constructed to the east of existing breakwater and the existing breakwater lengthened. The sand would be pumped through a submerged pipe only across the entrance to the proposed harbor, cutting down the distance through which the sand would have to be removed. Under this system the waves and currents would be relied upon to continue to move the sand to the east.

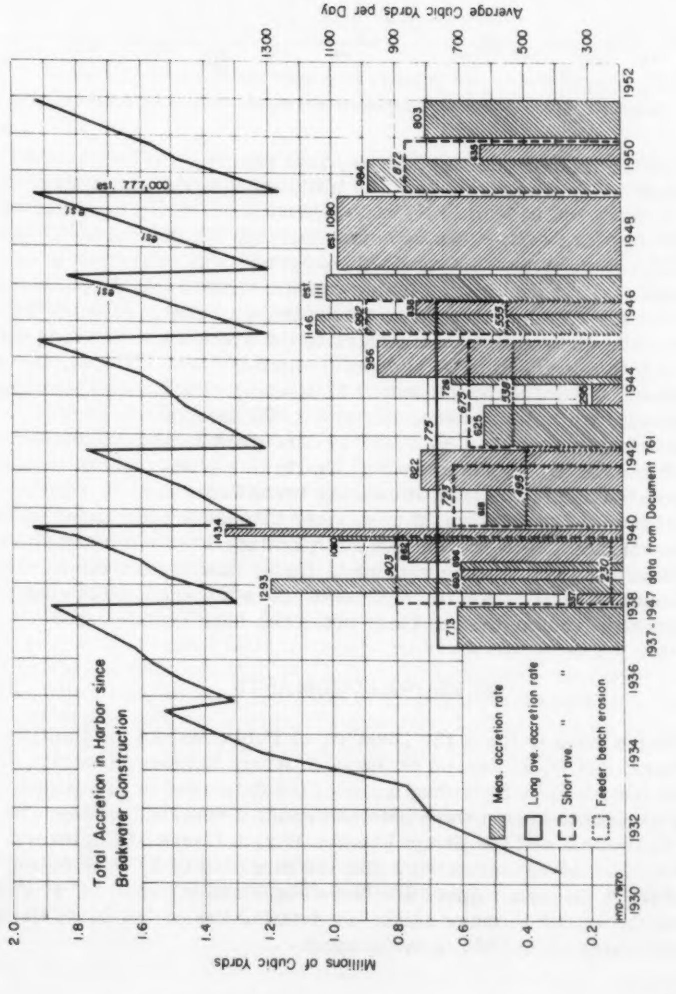
At the present time neither of these plans has been implemented.

ACKNOWLEDGEMENTS

The author wishes to thank the Director of Public Works for the City of Santa Barbara, and W. J. Herron of the U. S. Army Engineer District, Los Angeles for their help in providing many of the data used in this paper. Many of the data presented herein were obtained under a contract between the University of California and the Beach Erosion Board, Corps of Engineers, U. S. Army, the studies being conducted under the direction of J. W. Johnson, P. D. Trask, and W. N. Bascom. Special acknowledgement is due to M. P. O'Brien for allowing the author to make use of his detailed file on the Santa Barbara harbor which dates from 1935 to the present.

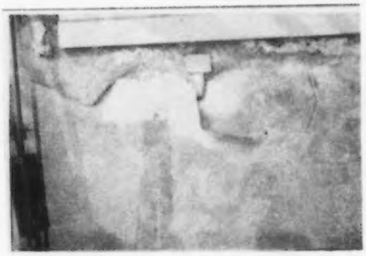
REFERENCES

1. Bascom, Willard N., Investigation of coastal sand movements, Santa Barbara, California, Part I, Institute of Engineering Research, University of California, Berkeley, Tech. Rept. Series 14, Issue 8, 38 pp, January, 1951 (unpublished).

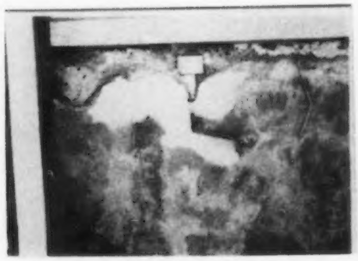


Accretion Rates of Sand Fill at End of Breakwater Inside Harbor, Santa Barbara, California

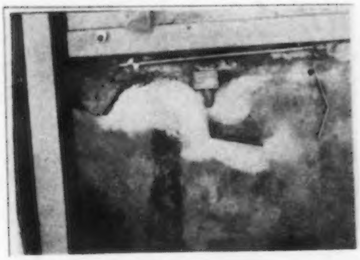
FIGURE 21



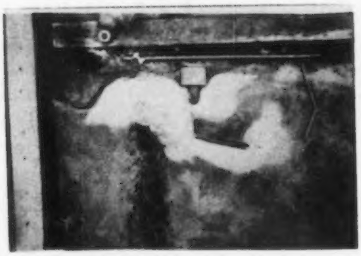
a. Run 7, End 420 min.
H = 0.0656 ft., T = 0.862 sec.



b. Run 7, End 480 min.

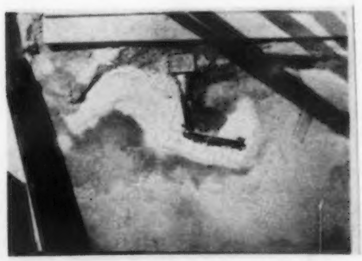


c. Run 7, End 600 min.



d. Run 7, End 780 min.

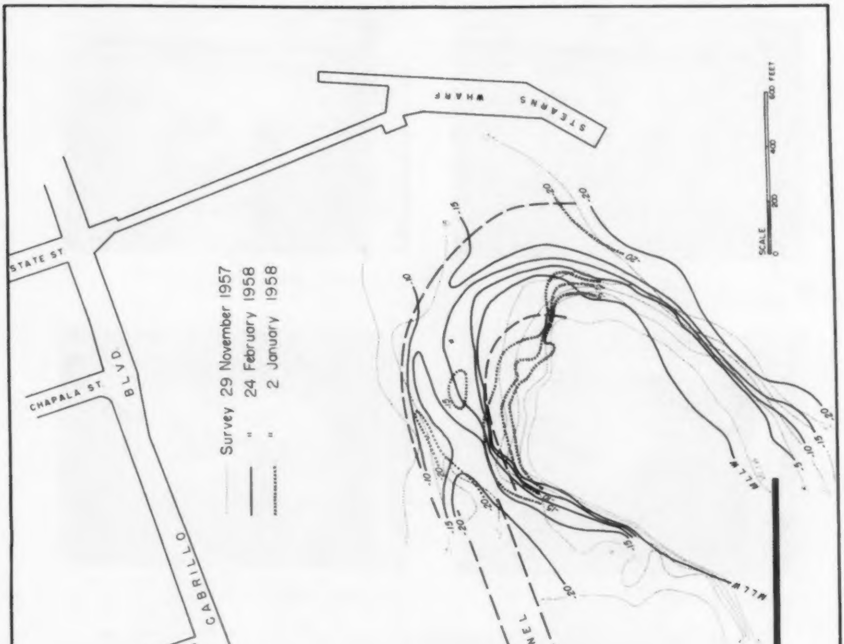
e. Run 7, End 840 min



f. Run 8, End. 990 min
H = 0.0494 ft., T = 0.862 sec.

Model Study of Sand Movement at Santa Barbara Breakwater
(After Lapsley, 1937)

FIGURE 22

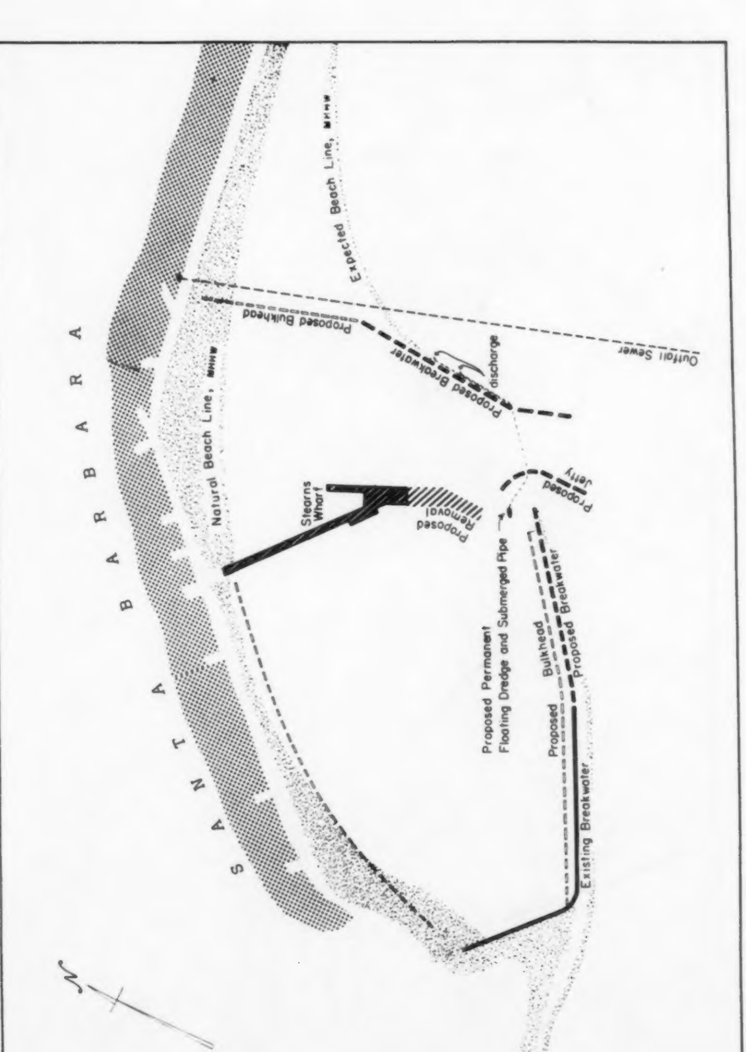


Effect of South East Storm on Sand Deposit at End of Breakwater, Inside Harbor, Santa Barbara, California

FIGURE 23


SANTA BARBARA HARBOR
CALIFORNIA
MAINTENANCE PLANS CONSIDERED
SANTA BARBARA CHANNEL
PACIFIC OCEAN
SANTA BARBARA
Mission Creek
(after U.S. Congress, 1941)
HYD-7998

FIGURE 24



Proposed Harbor Improvement, Santa Barbara, California (after Penfield, 1948)

FIGURE 25

2. Bascom, Willard N., The relationship between sand size and beach-face slope, Trans., Amer. Geophys. Union, Vol. 32, No. 6, pp 866-874, December 1951.

3. City of Santa Barbara, Public Works Department, Progress Report, harbor maintenance dredging program, 5 pp, May 1958 (unpublished).

4. Handin, John W., The source, transportation, and deposition of beach sediment in Southern California, U. S. Army, Corps of Engineers, Beach Erosion Board, Tech. Memo. No. 22, 125 pp, March 1951.

5. Johnson, J. W., M. P. O'Brien, and J. D. Isaacs, Graphical Construction of wave refraction diagrams, U. S. Navy Hydrographic Office, Pub. No. 605, 45 pp, Jan. 1948.

6. Johnson, J. W., Sand transport by littoral currents, Proc. Fifth Hydraulic Conference, Bulletin 34, State University of Iowa Studies in Engineering, pp 89-109, 1953.

7. Johnson, J. W., Dynamics of nearshore sediment movement, Bull. Amer. Assoc. of Petroleum Geologists, Vol. 40, No. 9, pp. 2211-2232, Sept. 1956.

8. Johnson, J. W., The littoral drift problem at shoreline harbors, Journal of the Waterways and Harbors, Div., ASCE, Vol. 83, Paper 1211, 37 pp, April 1957.

9. Lapsley, William W., Sand movement and beach erosion, M. S. Thesis in Civil Engineering, University of California, Berkeley, Calif., 54 pp, 1937, (unpublished).

10. O'Brien, M. P., Wave refraction at Long Beach and Santa Barbara, California, Bull. of the Beach Erosion Board, Corps of Engineers, U. S. Army, Vol. 4, No. 1, pp 1-12, Jan. 1950.

11. Penfield, Wallace C. and Tom D. Cooke, Report on a proposed improvement plan for Santa Barbara Harbor, Santa Barbara, California, The Santa Barbara Harbor Development Association, 81 pp, June 1948 (unpublished).

12. Shepard, F. P., Beach cycles in Southern California, Beach Erosion Board, Corps of Engineers, U. S. Army, Tech. Memo. No. 20, 26 pp, July 1950.

13. Trask, Parker D., Movement of sand around Southern California promontories, Beach Erosion Board, Corps of Engineers, U. S. Army, Tech. Memo. No. 76, 60 pp, June 1955.

14. Trask, Parker D., Source of beach sand at Santa Barbara, California, as indicated by mineral grain studies, Beach Erosion Board, Corps of Engineers, U. S. Army, Tech. Memo. No. 28, 23 pp, October 1952.

15. Trask, Parker D., Santa Barbara studies: Compilation of basic data, Inst. of Eng. Res., University of California, Berkeley, Tech. Rept. Series 14, Issue 2, 4 pp, plus figures, July 1952 (unpublished).

16. U. S. Congress, Beach erosion at Santa Barbara, Calif., House Document No. 552, 75th Congress, 3d Session, 51 pp, 18 March 1938.

17. U. S. Congress, Santa Barbara, C lif.: Letter from the Secretary of War, House Document No. 348, 77th Congress, 1st Session, 17 pp, Aug. 6, 1941.

18. U. S. Congress, Santa Barbara, Calif., Beach erosion control study, House Document No. 761, 80th Congress, 2nd Session, 35 pp, plus 13 plates, 22 December 1948.

19. U. S. Army, Corps of Engineers, Beach Erosion Board, Supplementary Report, beach erosion study at Santa Barbara, California, 15 pp, 18 Feb. 1942 (unpublished).

20. Wiegel, R. L., D. A. Patrick, and H. L. Kimberley, Wave, longshore current, and beach profile records for Santa Margarita River Beach, Oceanside, California, 1949, Trans. Amer. Geophys. Union, Vol. 35, No. 6, pp 887-896, Dec. 1954.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

THE EFFECT OF SEICHES AT CONNEAUT HARBOR

Ira A. Hunt,[1] A. M. ASCE and Leonas Bajorunas[2]

ABSTRACT

Conneaut Harbor has a record of unusually frequent accidents. This paper analyzes seiches on Lake Erie and shows that there is a correlation between water level fluctuation recorded at Buffalo and the accidents at Conneaut.

INTRODUCTION

The port of Conneaut, located in the extreme northeastern corner of Ohio on the southern shore of Lake Erie, has a record of unusually frequent accidents for which there are no apparent reasons. The Lake Carriers' Association and the Pittsburgh Steamship Division of the U. S. Steel Corporation have reported over 180 accidents during the period 1939 through 1951.

Second only to Cleveland among harbors on the Great Lakes in the receipt of iron ore and concentrate, Conneaut Harbor is man-made. Fig. 1 shows that it consists of an outer harbor sheltered by breakwaters and an inner harbor formed by the lower 3,000 feet of the Conneaut River and a slip built by the Pittsburgh and Conneaut Dock Company. The inner harbor is maintained by local interests and the slip is used to winter several of the large ore carriers of the U. S. Steel fleet.

The west breakwater is constructed of capped stone and from its outer end extends approximately 4,300 feet shoreward where there is a gap of 100 feet. The shore-arm breakwater has a structural length of 1,630 feet. The east breakwater extends approximately 3,700 feet shoreward from its light foundation located opposite and 600 feet from the head of the west breakwater. The outer 1,825 feet and inner 800 feet are of capped stone formation and the intermediate 1,050 feet are of timber-crib structure and concrete superstructure with stone riprap reinforcement along the lakeward side. There is

Note: Discussion open until November 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2067 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. WW 2, June, 1959.

1. Major, Corps of Engrs., U. S. Army.
2. District Engr., Chf., Special Studies, U. S. Lake Survey.

a 2,100-foot gap between the shore and the east breakwater. The U. S. Army, Corps of Engineers dredges the outer harbor to maintain depths of 20 feet in the westerly part and depths of 25 feet in the easterly part. All project depths and soundings are referred to Low Water Datum, elevation 570.5 feet above Mean Tide at New York City.

The two piers at the river mouth are 200 feet apart at the outer ends, gradually diverging to 350 feet apart at the inner end of the east pier. The piers are stone-filled timber-crib substructures with concrete superstructures. The channel between the piers and the turning basin has a controlling depth of 20 feet. Nearby, in prolongation of the improved channel, is the Pittsburgh and Conneaut Dock Company slip which is about 1,352 feet long and 166 feet wide with a least depth of 20 feet. The natural river channel bends sharply to the east from the turning basin.

Because of the numerous reports stating that strong currents and/or surges were the causative factor in many of the accidents, the U. S. Lake Survey was requested to investigate and determine whether or not such was the fact.

There are several possible sources of currents that might affect shipping entering the inner harbor. There are longshore currents generated by waves breaking at an angle to the shore line, general circulatory currents in Lake Erie, currents resulting from the outflow of Conneaut River, and currents resulting from seiches in Lake Erie. Surging action could be a result of local barometric disturbances, seiches, or resonance resulting from agreement between the natural period of the harbor and outside waves. The water surface within the harbor could be further disturbed by excessive wave reflections from the breakwaters and piers. This paper investigates seiches and their relation to currents and local surges and discusses briefly the design of the harbor as it pertains to wave reflections.

Analysis of Seiches on Lake Erie

A seiche is defined as inertial oscillations of water levels which persist after external forces have ceased to act. Lake Erie is renowned for the magnitude of its seiches. Wind and barometric disturbances are the external forces involved. However, barometric disturbances are so infrequent that they are not considered herein.

The free water surface of Lake Erie is subject to a shear stress as the wind blows over it, and the transfer of the energy of the wind to the fluid surface results in water surface displacement. The magnitude of the water surface displacement, or "wind set-up", is primarily a function of the depth of the lake, the wind speed, and the fetch length over which the wind blows. The maximum wind set-ups on Lake Erie occur at Buffalo because of strong southwesterly winds blowing down the long axis of the lake. Wind set-ups of 8.4 feet above mean lake level have been recorded at Buffalo and the recorded differences in water level elevations between the extremities of the lake along its long axis—Buffalo and Toledo—have been as high as 13.5 feet. When the wind subsides inertial oscillations result.

Seiches resulting from wind set-ups at Buffalo or Toledo will generate currents at Conneaut. In the paragraphs which follow, seiches resulting from wind set-up at Buffalo will be analyzed to determine if the currents generated by the oscillating action of the lake are large enough to be of concern to lake vessels utilizing Conneaut Harbor.

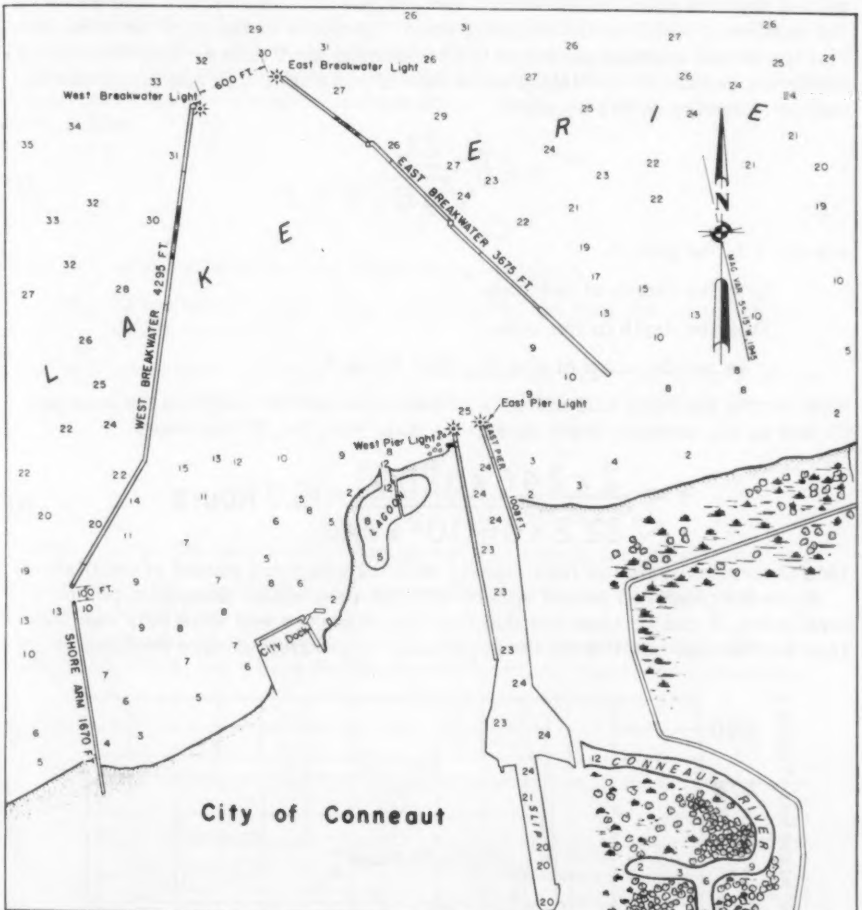


Fig. 1. Conneaut Harbor.

Seiches can be detected from recording water level gages. Gages at Toledo, Cleveland, Port Stanley, Port Colborne, and Buffalo were used in this study. A typical example of seiche action is depicted in Fig. 2. It can be seen that there is a definite periodicity to the water level fluctuations. The period depends upon the horizontal dimensions and depth of the lake and upon the number of nodes of the standing wave. Analysis of the gage records shows that the actual average period of the oscillation is 15 hours. The theoretical maximum period of oscillation for a lake of constant depth and cross-section can be computed by the equation

$$T = \frac{2L}{\sqrt{gD}} \tag{1}$$

where T is the period.

L is the length of the lake.

D is the depth of the lake.

g is acceleration of gravity, 32.2 ft/sec^2.

Substituting the right line distance of 246 miles as the length of the lake and 66 feet as the average depth along this right line, Eq. (1) becomes

$$T = \frac{2 \times 246 \times 5280}{\sqrt{32.2 \times 3600^2 \times 66}} = 15.7 \text{ hours} \tag{2}$$

This theoretical value agrees closely with the observed period of oscillation.

Since the observed period agrees with the theoretical maximum period of oscillation, it can be expected that the standing wave will have only one node. This fact is also verified by the gage data. The water surface profiles at the

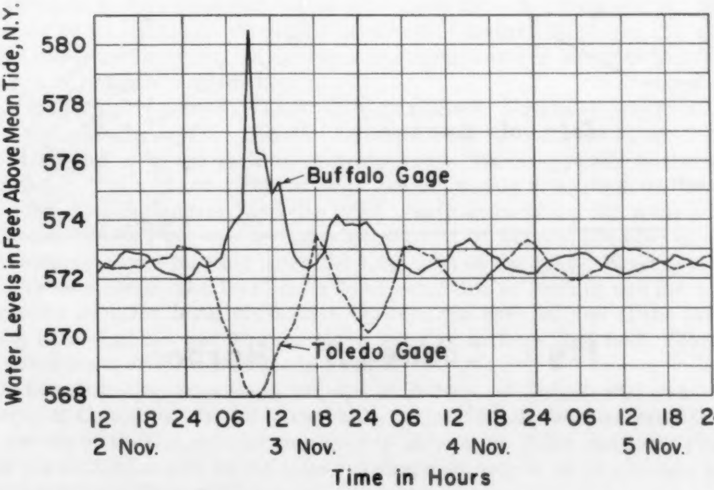


Fig. 2. General Lake Erie seiche resulting from storm of 3 Nov. 1955.

time of maximum wind set-up and for several subsequent intervals were determined for a number of seiches. Fig. 3 presents the water surface profiles for a typical seiche occurring on Lake Erie; note that the standing wave definitely oscillates about one nodal point. It is interesting, too, that the location of this nodal point does not vary appreciably for different seiches.

The amplitudes of the oscillations of a seiche diminish with time. The maximum amplitudes of a seiche on Lake Erie occur at Buffalo. The reduction in the height of the water displacement should be a decay type function. A decay curve for the maximum amplitudes of each successive standing wave can be written

$$B = B_0 \, e^{-.86n} \tag{3}$$

where B is the rise at Buffalo above mean lake level.

B_0 is the height above mean lake level of the wind set-up which induces the seiche.

n an integer, is the number of the standing wave and is equal to the time subsequent to the maximum wind set-up divided by the period, i.e.

$$n = \frac{T}{15} \tag{4}$$

Table 1 gives the values of $\frac{B}{B_0}$ for successive maximum rises at Buffalo and shows that the amplitude of the seiche diminishes rapidly with each subsequent oscillation.

Often the winds continue to blow after the maximum set-up has been reached at Buffalo, but with a much reduced force, so that there continues to

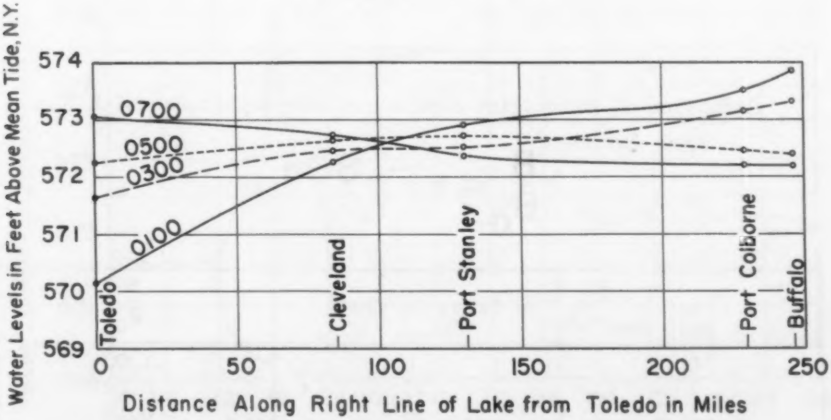


Fig. 3. Water level profiles during seiche of 4 Nov. 1955.

be a small water surface displacement. Although a small wind set-up persists, the force of the wind is reduced sufficiently to permit the occurrence of inertial oscillations. Eq. (3) was derived from actual gage records which indicate fluctuations from mean lake level resulting from wind set-up and seiches combined. An example of the combination of the two phenomena is illustrated by the water level displacements at Buffalo and Toledo shown in Fig. 4. In this case, the water level at Buffalo remained above mean lake level for an extended period.

Although Eq. (3) was derived from Buffalo data, it has been found to be equally applicable to water level displacements recorded at Toledo.

The horizontal displacement of water particles varies periodically. The differential equation of motion has been simplified by Defant[1] to read:

$$\Delta \pi = \frac{4\pi^2}{g T^2} \xi \Delta X \tag{5}$$

Rearranged, it is

$$\xi = \frac{\Delta \eta}{\Delta X} \quad \frac{g T^2}{4\pi^2} \tag{6}$$

where ζ is the horizontal displacement in feet.

$\frac{\Delta \eta}{\Delta X}$ is the instantaneous water surface slope at a given locality.

T is the period of oscillation in hours.

g is the acceleration of gravity in $^{ft}/hrs^2$.

If the water surface slope is expressed in feet per mile and the period of oscillation taken is 15 hours, then

$$\xi = 450{,}000 S \tag{7}$$

TABLE 1

Reduction of Successive Maximum Seiche Amplitudes with Time

$$\frac{B}{B_o} = e^{-.86n}$$

n	Time, in Hours	$\frac{B}{B_o}$
0	0	1.00
1	15	0.42
2	30	0.18
3	45	0.08
4	60	0.03
5	75	0.01

and

$$\frac{d\xi}{dt} = 450{,}000 \frac{dS}{dt} \tag{8}$$

The horizontal surface velocity of a particle can be determined by measuring the rate of change in the surface slope at Conneaut. The rate of change in slope can be measured from surface profile plots such as those shown in Fig. 3, p. 35. If the water surface is considered to be a plane which tilts about the nodal point, the rate of change in slope can be approximated by measuring the rate of water surface displacement at Buffalo and dividing it by the distance from the nodal point to Buffalo. For convenience the time interval chosen is $\frac{T}{2}$; the period of maximum oscillation at Buffalo. Some results are given in Table 2. The values determined by considering that the lake tilts as a plane are very close to the results obtained by drawing tangents to the observed water surface profiles.

The average horizontal velocity in the vicinity of Conneaut can be determined by substituting for $\frac{dS}{dt}$ in Eq. (8).

If

$$\frac{dS}{dt} = \frac{\Delta B}{138 \times 7.5 \times 3600} \tag{9}$$

then

$$\frac{d\xi}{dt} = 0.12 \, \Delta B \tag{10}$$

where B is the maximum water surface displacement at Buffalo during a time interval, $\frac{T}{2}$.

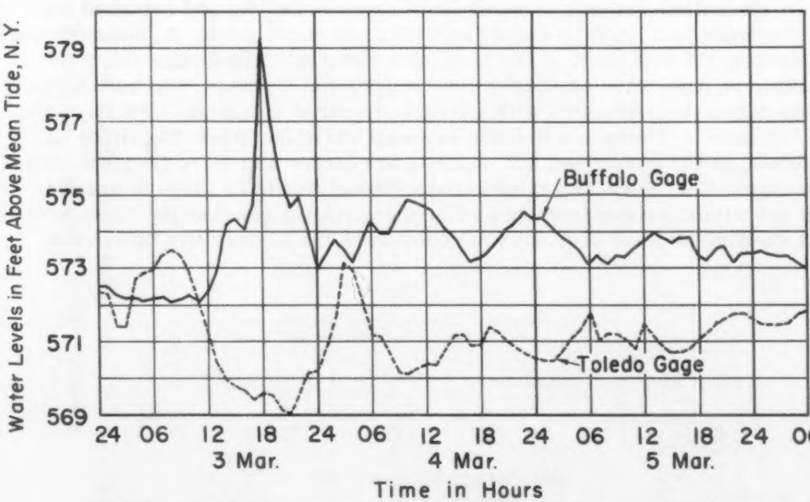


Fig. 4. Combination of seiche and wind set-up during period 3-6 Mar. 1954.

The rate of change of water levels recorded at various gage sites on Lake Erie during seiches is not constant. The ratio of the maximum rate of change during one hour to the average rate of change at selected sites is

Buffalo	2.3
Erie	2.5
Port Stanley	1.9

and a reasonable assumption for this ratio in the vicinity of Conneaut is 2.2. As a result, the maximum horizontal velocity at Conneaut for a one-hour period can be computed from the equation

$$\left(\frac{d\xi}{dt}\right)_{max} = 2.2\ (0.12\ \Delta B) = 0.26\ \Delta B \tag{11}$$

There is no distinct pattern between the time of the maximum rate of change of water levels at Buffalo and the time of maximum water level.

It is not unreasonable to expect currents of approximately one foot per second outside of Conneaut Harbor as a result of seiches and the currents at the entrance to the inner harbor could be higher. Although not appreciable in themselves, these seiche currents, when added to the surface currents generated by the wind and the longshore currents, could cause accidents at Conneaut. The entrance to the inner harbor is only 200 feet wide. Theoretically, a cross-current of 0.8 foot per second could cause one of the large ore carriers, 600 feet long with a 60-foot beam, entering the center of the inner harbor to strike the edge of a pier. Since the helmsman would attempt to compensate for the movement of the boat, it would take a cross current greater than 0.8 ft/sec to cause an accident.

Correlation Between Water Level Fluctuation Recorded at Buffalo and Accidents at Conneaut

A statistical analysis of water level rises at Buffalo and reported accidents at Conneaut was made to see if there was any correlation. A frequency curve indicating the frequency of the maximum water surface displacement at Buffalo on days when accidents were reported at Conneaut was calculated. This curve was compared with a frequency curve of non-accident days chosen at "random". There is a definite seasonal variation in the magnitude of seiches; in the spring and fall, seiches are larger and more frequent than in summer. Therefore, the random days chosen should be close to accident days but not within the continuation of the seiche. After considering these factors, the random accident-free days selected were the seventh day before the

TABLE 2

Relationship of Water Surface Slope at Conneaut with Water Level Fluctuation at Buffalo

Date of Seiche	Change in Slope at Conneaut, Feet per Mile	Fluctuation at Buffalo, in feet	Distance, Buffalo to Node, in Miles	Derived Ratio Fluctuation to Distance, Feet per Mile
21 February 1953	0.025	3.76	133	0.028
3 March 1954	0.032	5.50	140	0.039
4 November 1955	0.011	1.69	145	0.012
4 April 1956	0.018	2.45	131	0.019
24 April 1958	0.026	3.65	140	0.026
Average	0.022	3.41	138	0.025

accident and the seventh day after the accident. Because the number of reported accidents is quite high, the effect of chance observations is greatly reduced and the results are quite reliable.

A comparison of the two frequency curves, Fig. 5, shows that there is a rather large differential in themagnitude of lake level fluctuations at Buffalo for accident days over random days. If there were no correlation between stage fluctuations at Buffalo and accidents at Conneaut, the curves should be approximately the same. Therefore, it is highly probable that seiches in Lake Erie contribute to the number of accidents in the harbor at Conneaut.

The hypothesis that seiche currents have appreciable effects on boats in the harbor was further strengthened by conversations with masters of lake carriers, captains of tug boats, and local inhabitants in Conneaut. One vessel master was fully aware of the "tilting" of Lake Erie, the period of oscillations, and the currents which result therefrom. Another, Captain Richer of the Great Lakes Towing Company, told of being warned by the master of a freighter which had docked during the early hours of the morning that bad currents from the east had made it extremely difficult for him to control the boat while entering the harbor. Several hours later, when Captain Richer went out to bring a vessel into the harbor with his tugboat, he tried to make allowances for these strong currents, only to find that there were none.

Transitory currents, such as those described by Captain Richer are of the type current caused by seiche action which lends credence to the belief that seiches are the cause of the currents peculiar to Conneaut Harbor.

Most important when considering cross currents at the entrance to the inner harbor are the seiches which result from set-ups at Buffalo and Toledo. However, seiches resulting from water surface displacements because of north-south winds cause appreciable damage to boats at berth in the inner harbor and the slip. Frequent accident reports cite surging action as the cause of damage. The layout of the breakwaters makes the inner harbor most susceptible to large fluctuations caused by seiches oriented in the north-south direction.

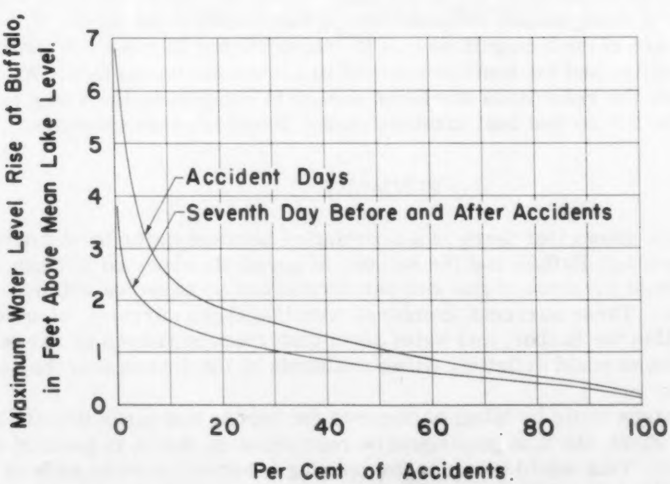


Fig. 5. Frequency of maximum water level set-ups at Buffalo.

Sub-surface current observations over two-day periods were made in the outer harbor by the U. S. Army Engineer District, Buffalo, in 1944 and 1949. These observations indicated that there is a definite circulatory pattern of currents existing inside the harbor. The current velocities varied from 0.2 to 1.0 foot per second, with the currents at the entrance to the inner harbor approximately 0.5 foot per second. Surface current patterns, were observed to exist even on days with little wind, at which time there was a substantial current flowing eastward through the gap in the west breakwater and a strong current flowing into the opening of the inner harbor. These currents which are probably caused by general circulatory currents in Lake Erie, are definitely detrimental to navigation when compounded with seiche currents—although in themselves they are not troublesome.

Wave reflections from the breakwaters and the vertical bulkheads of the piers further complicate the picture. The breakwaters for the most part are of capped stone with a harbor-side slope of 1:1.3. The prevailing winds are from the northwest. The orientation of the east breakwater is such that the waves enter the outer harbor because it extends farther lakeward than the west breakwater. The maximum waves to be expected have a significant wave height (H) of 8 feet and a significant wave period (T) of 6 seconds. According to Hunt in the Design of Seawalls and Breakwaters,(2) to minimize wave reflections, the slope of the structure (i) should be such that

$$i^2 < \frac{H}{T^2} \tag{12}$$

For the storm waves expected at Conneaut the harbor slope of the breakwater should be flatter than 1:2.1

$$i < \left(\frac{8}{6^2}\right)^{\frac{1}{2}} = \frac{1}{2.1} \tag{13}$$

With a slope as steep as 1:1.3, the large waves that enter the outer harbor are capable of being totally reflected by the breakwaters and piers. These reflections are of such magnitude that the outer harbor is rendered useless in heavy weather and the boats are forced to anchor out on the lake. With normal winds the reflections are large enough to complicate the water surface within the harbor so that boat masters cannot detect adverse currents.

SUMMARY

It has been shown that there is a correlation between water level fluctuations recorded at Buffalo and the number of accidents reported at Conneaut Harbor. Seiche currents of one foot per second can be expected within the outer harbor. These currents, combined with longshore currents, circulatory currents within the harbor, and water level disturbances caused by excessive wave reflections could definitely cause accidents at the entrance of the narrow inner harbor.

Definite steps could be taken to improve the harbor and make it safer for navigation. First, the east pier could be reoriented so that it is parallel to the west pier. This would increase the opening between the outer ends of the piers from 200 feet to 350 feet. Such a reorientation is the most important single construction that could be undertaken to eliminate accidents. Second,

the east breakwater could be extended to the shore. This would eliminate longshore currents and reduce seiche currents and would greatly reduce the circulatory cross currents at the entrance of the inner harbor. With the elimination of the longshore currents from the east, the littoral sand drift would also be eliminated thereby reducing somewhat the maintenance dredging which is now required. Third, the ends of the breakwater could be altered so that the east breakwater does not extend farther north than the west breakwater. The 600-foot gap between the breakwaters is sufficient, and, in any reorientation of the breakwaters, this gap should not be materially increased. Reorientation of the breakwaters would do much to prevent the entrance of large waves into the outer harbor. There is nothing economically feasible that can be accomplished to eliminate wave reflection once the waves have entered the outer harbor.

The Waterways Experiment Station, utilizing some of the findings of this study, is constructing a hydraulic model of Conneaut Harbor. This model will introduce transverse currents as well as waves and experimentation will determine what harbor improvements are necessary to make Conneaut Harbor safe for navigation. When considering marine structures which may well cost several millions of dollars, the cost of conducting model studies is a small percentage of the over-all costs and may well save hundreds of thousands of dollars by insuring proper design. However, a thorough technical investigation is always necessary first to insure that the correct data and techniques are utilized in the hydraulic model. It will be interesting to see if the conclusions reached in this analysis of Conneaut Harbor are substantiated by model tests.

REFERENCES

1. H. V. Sverdrup, M. W. Johnson, and R. H. Fleming, The Oceans (1942) pp. 539-540.

2. Hunt, Jr., I. A., Design of Seawalls and Breakwaters, U. S. Lake Survey, Corps of Engineers, January (1958).

P

St

Sul
Term
subsi
sulta

In
Pedr
a ver
move
Tl
prese
ing r
area
Tl
of pe
possi
Inasr
the p
its r
surf
T
highl
port
vers

Note
a
pa
of
1. A

Journal of the

WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

SUBSIDENCE PROBLEM IN THE LONG BEACH HARBOR DISTRICT

R. F. Berbower,[1] A. M. ASCE

ABSTRACT

Subsidence of the ground surface due to oil extraction is not new, but at Terminal Island, California, it has set a new record. The history of this subsidence is given, studies to determine the cause are described, and resultant problems and solutions are discussed.

Description of Effects

In the southern part of Los Angeles County, contiguous to and beneath San Pedro Bay and extending over several incorporated and unincorporated areas, a very extensive and deep-seated land subsidence, accompanied by horizontal movements, has developed over the past twenty years.

This subsidence has exceeded all early predictions such that it now presents a difficult and expensive problem of effectively expediting and financing remedial work. However, the overall economics of the affected surface area dictate that every possible effort be devoted to corrective measures.

This particular land subsidence has been reliably attributed to the recovery of petroleum and the resultant reduction of fluid pressures in certain and possibly all of the producing zones of the Wilmington (California) Oil Field. Inasmuch as oil and gas have each been produced in this field, as is usual in the process of producing oil, it has been concluded that each withdrawal exerts its respective influence in altering the virgin soil characteristics and subsurface pressures.

The subsidence is occurring in an area which is intensively developed with highly valued surface and subsurface improvements which are extremely important to the economy of the immediate area and its surroundings. The adverse effects are greatest in uplands and filled submerged land areas adjacent

Note: Discussion open until November 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2068 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. WW 2, June, 1959.

1. Asst. Chf. Harbor Engr., Long Beach Harbor Dept., Long Beach, Calif.

to San Pedro Bay where the ground elevation prior to land subsidence was only a few feet above extreme high tides of the Pacific Ocean. These lands, now below high tides except where raised, are the most subject to flooding. Segments of Los Angeles Harbor, all of the developed portions of the Port of Long Beach and the lower reaches and outlet of the Los Angeles River—the principal flood control channel of Los Angeles County—are located within the subsiding area.

To illustrate the extent of the potential flooding possibilities, the total existing waterfrontage that has subsided in excess of two feet and up to a maximum of 25 feet is approximately 30 miles. In addition to this 30 miles, there are four miles of waterfront in a more slowly sinking area—principally southwest of the elliptical shaped subsidence bowl—and some three miles to the east of the present large disturbances, which are expected to subside appreciably.

Status in 1958 and Resulting Picture

In a few instances was the original land or structure elevation adjacent to the waterfront more than 6.5 feet above the predicted extreme high tides, and in many locations much less. The highest tides occurring in the area are predicted as 7.5 feet above mean lower low water and are comparatively free from excessive variations.

Occasionally exceptions of a foot or more result from seasonal offshore storms. These overages usually threaten when coinciding with high tides. Conditions in the affected area are such that a strong earthquake might reduce some filled land elevations in significant amounts.

The occurrence of long period seismic sea waves or tsunamis* as a result of seismic disturbances in the North Pacific have been known to add substantially to the level of the adjacent bay waters for short rhythmic periods. Since the waves can persist for many hours, such occurrences coincident with normal high tides, present serious flooding hazards when levees are low or unstable. The period of the tsunamis of November 1952 approximated 34 minutes in the local harbor with a height of about four feet from trough to crest.

Physical improvements in close proximity to the waterfront and at low elevations are valued in excess of five hundred million dollars, including some 2,500 producing or salvageable oil wells with their many accessory structures such as pipelines, tank farms, etc. Back area developments not located immediately adjacent to the waterfront, but affected in other respects and in some instances subject to possible flooding, represent additional high valued investments.

Essentially all of the affected area is utilized for transportation, industry, commerce and oil. Thus the area is important to a very large population—in fact, much of the population of the Pacific Southwest. Some of the subsidence area is beneath San Pedro Bay, where the water merely becomes deeper. Much of it is beneath the waters of the channels, slips, basins and fairways of the harbors and beneath the Flood Control Channel which serves to protect the

*"The Tsunami of November 4, 1952 as Recorded at Tide Stations." Special Publication No. 300, U. S. Department of Commerce, Coast and Geodetic Survey, Washington, D. C.

adjacent lands. To the extent that it relieves certain dredging obligations, subsidence is an asset in these water areas.

By 1958, in excess of 2,000 acres of natural or artifically created industrial land which was above the level of high tide before subsidence had settled well below that level or would have been below that level had it not been for remedial work theretofore accomplished.

The normal ground water table is about elevation plus 3.5 feet above mean lower low water, and inland areas below that elevation would naturally become brackish lakes even though protected from ocean tides. The great scope of the remedial program to compensate for subsidence can thus be seen.

The rate of subsidence per annum varies a great deal as to time and place but is of such magnitude as to impose important time limits on planning, scheduling and constructing the remedial works.

Fig. 1, following, illustrates with heavy lines the isobases of equal subsidence ranging from two feet to 25 feet, over a period from 1928 to August 1958.

The bowl of subsidence by August 1958 had become sufficiently large that the limits within which two feet or more of subsidence had occurred included over fifteen square miles. The area shown within the roughly shaped ellipse defined by the two foot isobase on Fig. 1 is about 6.0 miles along the major axis and nearly 4.0 miles along the minor axis. The total volume of subsidence represented by the portion of the subsidence bowl within the two-foot isobase is approximately 60 million cubic yards. If it were feasible to completely restore original land surfaces, within this two-foot mark, to the levels which would have existed without subsidence, it would require some 50 million cubic yards of compacted earth fill.

Subsiding areas outside the two-foot isobase are considerably greater than those within as is the volume of subsidence. Since there is general subsidence of small magnitude in other nearby areas, it is difficult to define the exact limits of the area of influence.

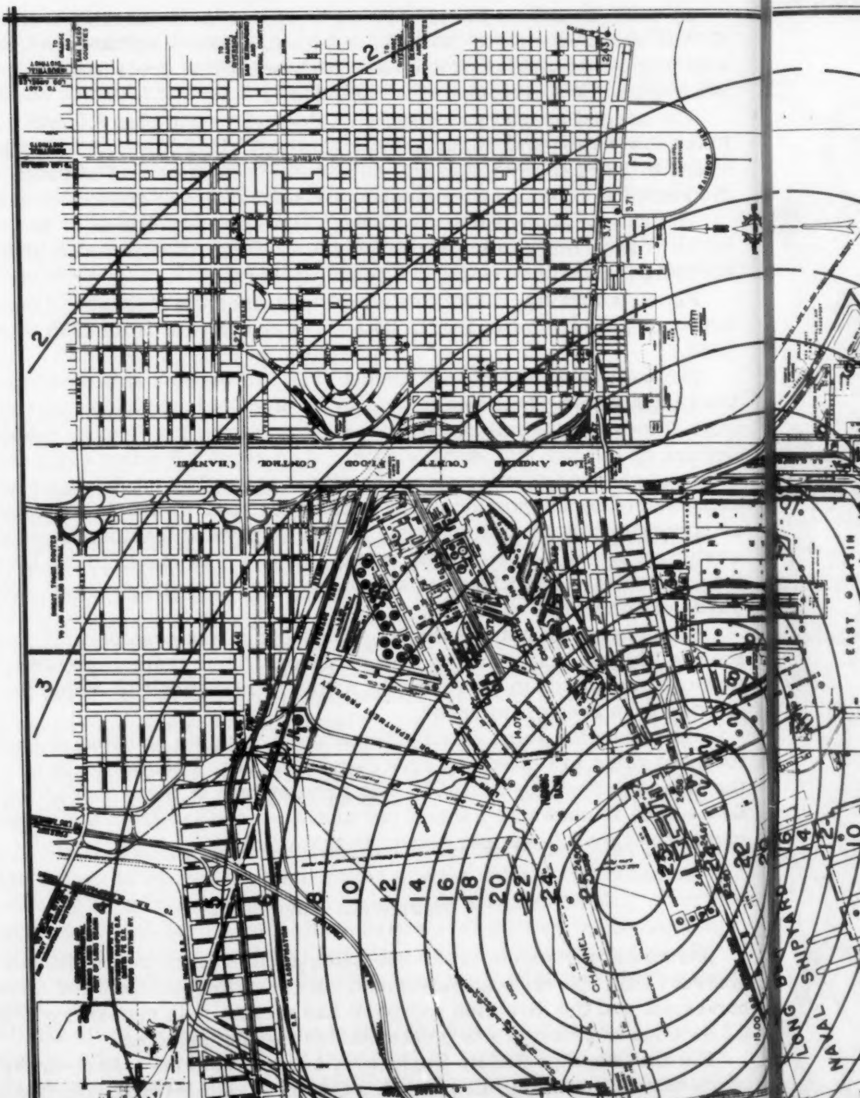
Since changing rates of subsidence are a valuable planning and analysis tool, the rates are observed very carefully. Fig. 2 shows the yearly rates for the period ending August 1958. The present rate at the center of the subsidence bowl is shown as 1.0 feet per annum—a decline from the recent maximum of nearly 2.5 feet per annum at that point.

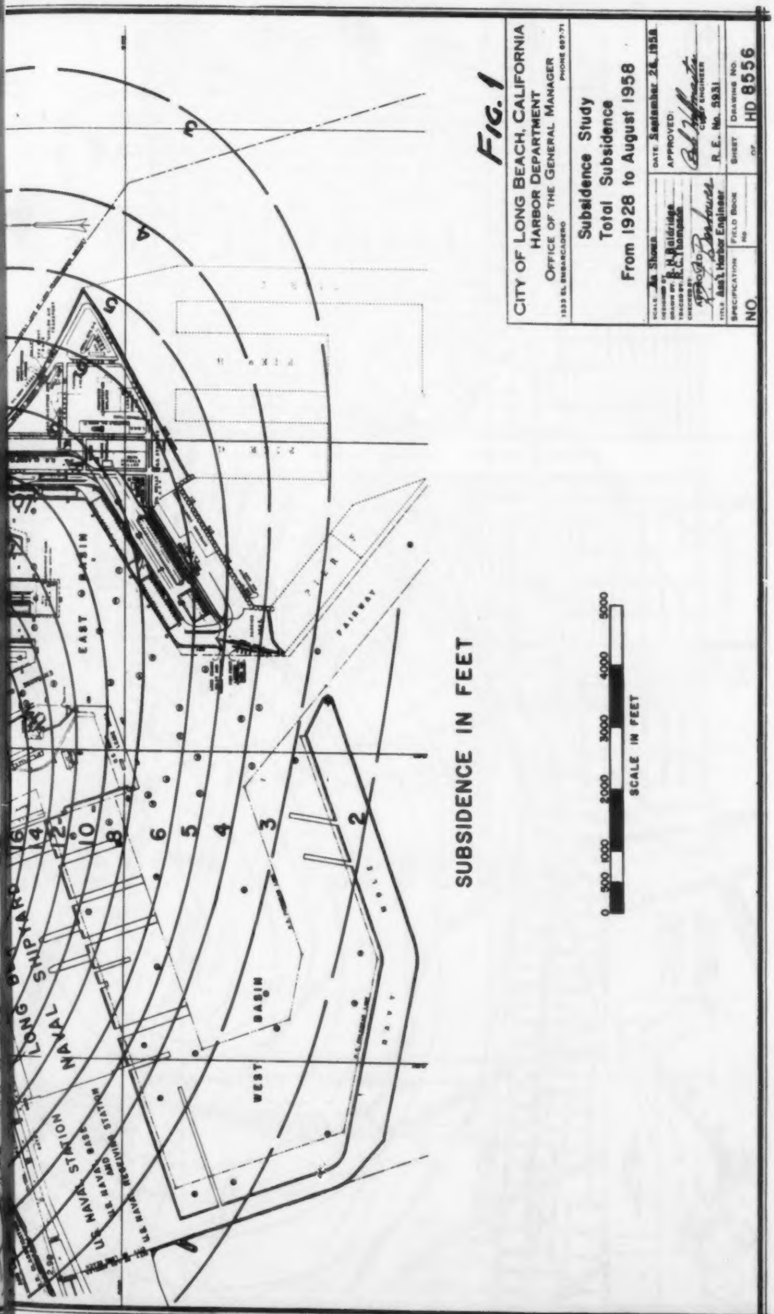
Horizontal Movements

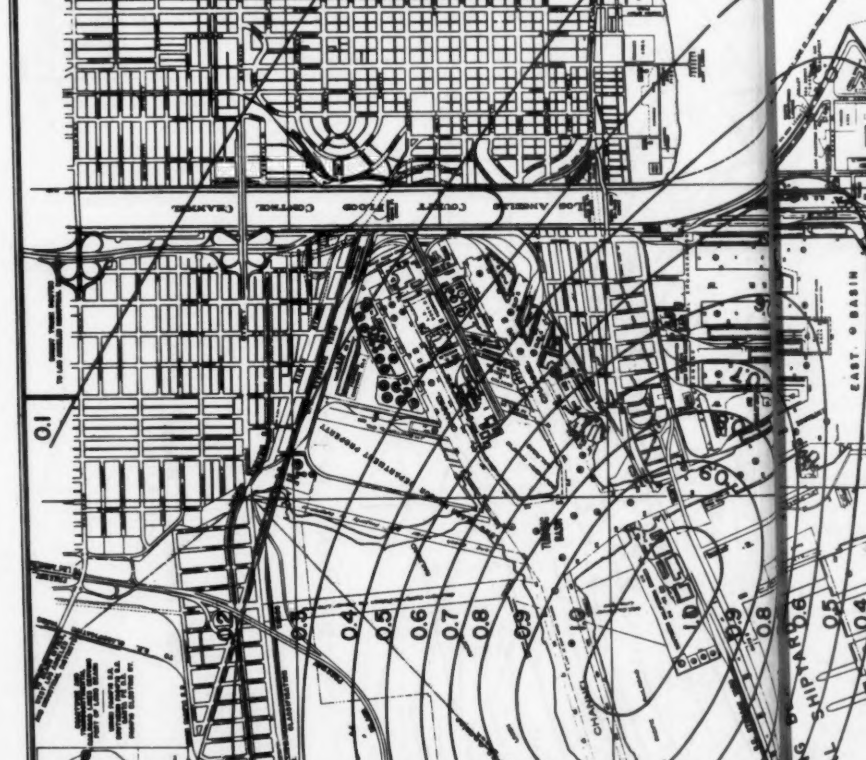
Horizontal movement in the subsiding area is much more difficult to measure than the vertical movement. In order to determine the amount of movement and the direction any point has moved, it is necessary to determine its location in reference to points outside the moving area.

The Los Angeles County Engineer's office maintains a first-order triangulation network covering Los Angeles County. This network consists of many triangulation stations and the geographic position of each station is very accurately located.

When it is necessary to determine the accurate location of any other point, or points, a small secondary triangulation network is established within the primary first-order triangulation network and the position of each point is determined.


LOS ANGELES COUNTY FLOOD CONTROL CHANNEL
EAST BASIN
CERRITOS CHANNEL
LONG BEACH NAVAL SHIPYARD
NAVAL STATION
VICINITY SKETCH
2
3
8
10
12
14
16
18
20
22
24
25


CITY OF LONG BEACH, CALIFORNIA
HARBOR DEPARTMENT
OFFICE OF THE GENERAL MANAGER
Subsidence Study
Total Subsidence
From 1928 to August 1958
DATE September 24, 1958
HD 8556
FIG. 1
SUBSIDENCE IN FEET
SCALE IN FEET
EAST BASIN
WEST BASIN
LONG BEACH NAVAL SHIPYARD
U.S. NAVAL STATION


EAST BASIN
LONG
NAVAL
SHIPYARD
NAVAL STATION
CHANNEL
CERRITOS
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

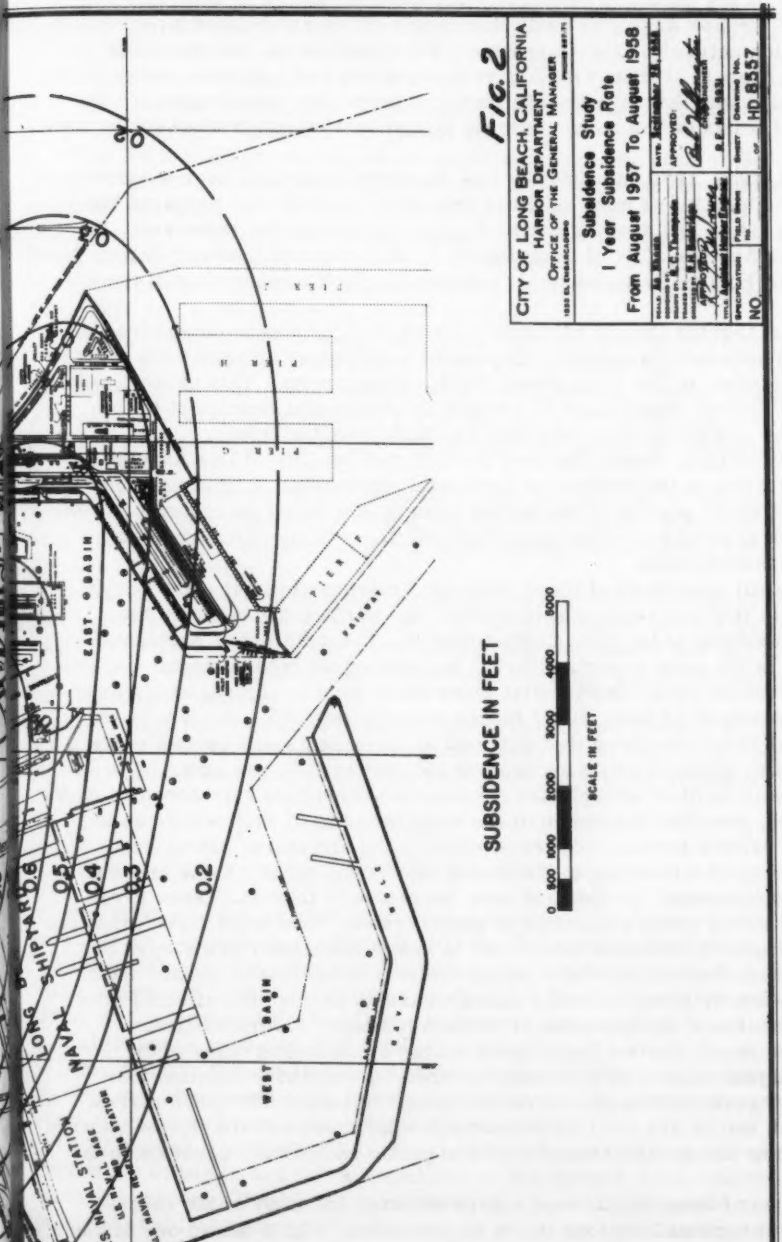


FIG. 2
CITY OF LONG BEACH, CALIFORNIA
HARBOR DEPARTMENT
OFFICE OF THE GENERAL MANAGER
Subsidence Study
1 Year Subsidence Rate
From August 1957 To August 1958
HD 8557
SUBSIDENCE IN FEET
0 500 1000 2000 3000 4000 5000
SCALE IN FEET
PIER F
PIER G
PIER H
EAST BASIN
WEST BASIN
FAIRWAY
LONG BEACH NAVAL SHIPYARD
US NAVAL STATION
0.2
0.3
0.4
0.5
0.6
0.8

If this secondary network is rechecked at a later date, the amount of movement of any point and the direction it has moved can be determined.

In 1937 the Los Angeles County Surveyor's office established a secondary triangulation network in the Long Beach-Wilmington area, and from this network Mr. Francis M. Bates made a very extensive and accurate survey of the Union Pacific and Southern Pacific Railroad properties and determined the location of points on the many concrete monuments he established during the survey.

In 1949, 1950, 1951 and 1954 the Los Angeles County Engineer's office rechecked the location of many of these survey monuments and prepared the maps (Fig. 3A to 3E) showing the past history of horizontal movement. The City of Los Angeles, City of Long Beach, U. S. Coast and Geodetic Survey and Long Beach Harbor Department also cooperated in the rechecking of these points.

The Los Angeles County Engineer's office is preparing to determine the location of several triangulation stations in a secondary triangulation network being established by the Long Beach Harbor Department. This network will enable the Harbor Department to recheck the horizontal movement in the harbor area at shorter intervals than has been possible before.

The City of Long Beach Engineer's office and the City of Los Angeles are also cooperating in the recheck of horizontal movement. A photogrammetric survey of a small portion of the harbor area is also being incorporated to determine the accuracy of using photogrammetric methods of horizontal and vertical measurements.

The overall magnitude of these horizontal movements is at many points measured in feet and reduces substantially the surface areas within the boundary markers of large property holdings. The differential horizontal movement is the most important factor in planning surface remedial work or new construction work. Differential movements have in certain locations attained a magnitude of some 4-1/2 inches or more per 100 feet, or 0.37 per cent. It has been calculated that differential movements will probably attain magnitudes of as much as six inches per 100 feet or 0.50 per cent.

The elastic limit of all ordinary construction materials can therefore easily be exceeded, provided the design of the installation is of such nature as to permit the earth's forces to be transmitted to the structure. Great local concentrations of stress due to horizontal movement occur. Some areas experience compression, torsion and even reversals to tension. Other areas experience initial tension followed by compression. In at least three instances where the capacity of subsurface strata to resist the subsurface shears has been exceeded, these strata have failed suddenly in horizontal shear. Such failures distort or shear oil wells casings passing through the affected strata and also result in a readjustment of surface positions in limited area.

The Long Beach Harbor Department makes a continuous study of the present and past history of the subsiding area. Every three months, the Harbor engineers recheck the elevation of approximately 550 bench marks. These bench marks are used for construction purposes and also for subsidence information by the Harbor Department and public and private agencies in the area.

Thirty-two of these bench marks were selected for study of the rate of subsidence at various locations in the Harbor Area. Fig. 4 shows one of these bench marks near the center of the bowl of subsidence. The curve shows the rate per year in feet this bench mark has subsided as plotted every three

months since November 1946. The yearly subsidence rate increased to 2.37 feet for the period November 1950 to November 1951 and then decreased to 1.45 feet from November 1953 to November 1954. In January 1955, a subterranean earthquake occurred in the area with subsurface horizontal movement that damaged approximately 75 oil wells. The yearly rate again increased to 1.69 feet for this bench mark from May 1954 to May 1955, and then decreased to 1.01 feet from August 1957 to August 1958.

Figs. 4A and 4B shows the actual subsidence rate per year of Bench Mark No. 8772 plotted on semi-logarithmic paper and estimated subsidence rates for total subsidence of 29 feet and 35 feet. These graphs are used by the Engineering Division to study the possible total subsidence at Bench Mark No. 8772, which is very near the center of subsidence. As long as the actual subsidence rate per year remains below the 35 foot total line the total amount to be expected in the center is 35 feet or less.

A comparison of the average daily oil production and the yearly subsidence rate of Bench Marks No. 700 and No. 8772 are shown in Fig. 6. The average daily total oil production includes flowing wells and wells being pumped. The average daily pumped production curve includes only production from wells that are pumped. The production figures are for the entire Wilmington Oil Field.

Fig. 7 shows the areas that have been affected by local earthquakes at various times. These earthquakes have damaged oil wells at depths of 1,500 feet to 2,500 feet below the surface and a large number of the wells have been damaged beyond repair.

Changes in elevation of bench marks during the local earthquake of November 17, 1949, are shown in Fig. 8. Approximately 302 oil wells were damaged at this time and 194 were damaged beyond repair and had to be abandoned. The estimated cost of remedial work and redrilling of these wells was 16 million dollars.

The changes in elevation of bench marks during the January 25, 1955 local earthquake are shown in Fig. 9. Approximately 75 oil wells were damaged by this earthquake and a very large percentage of them were damaged beyond repair.

Subsidence Studies

The first reports on subsidence in the Long Beach Harbor Area were made in 1945. One report by Frederic R. Harris, Inc. to the Chief of the Bureau of Yards and Docks, Navy Department, predicted a total ultimate subsidence of about seven feet. The other report by James Gilluly, Harry Johnson and U. S. Grant to the Board of Harbor Commissioners, City of Long Beach, predicted a total ultimate subsidence of about nine feet.

All available information on subsidence in Long Beach and other areas was studied and discussed in the Harris and Grant reports. Laboratory and field tests were made to provide additional information.

In May 1948, the Harbor Subsidence Committee was formed by representatives of public and private agencies in the Harbor Area, and this committee appointed a Technical Committee, composed of personnel and consultants of public and private agencies, to study the subsidence problem in the Long Beach Harbor Area. The general opinion of the Technical Committee was then that a total ultimate subsidence of 16 to 18 feet could be expected.


DOMINGUEZ CHANNEL
0.80
1.00
1.20
1.40
1.60
1.80
2.00
2.50
3.00
WIL. C-1
WIL. D-1A
WIL. D-1
WIL. E-1
WIL. F-1
COM. F-12
COM. G-12
CITY
LOS
ANGELES
CHANNEL
WIL. G-1
WIL. C-2
WIL. D-2
WIL. E-2
WIL. F-2
WIL. F-2A
WIL. G-2
WIL. H-2
DOMINGUEZ
SEE
SHEET

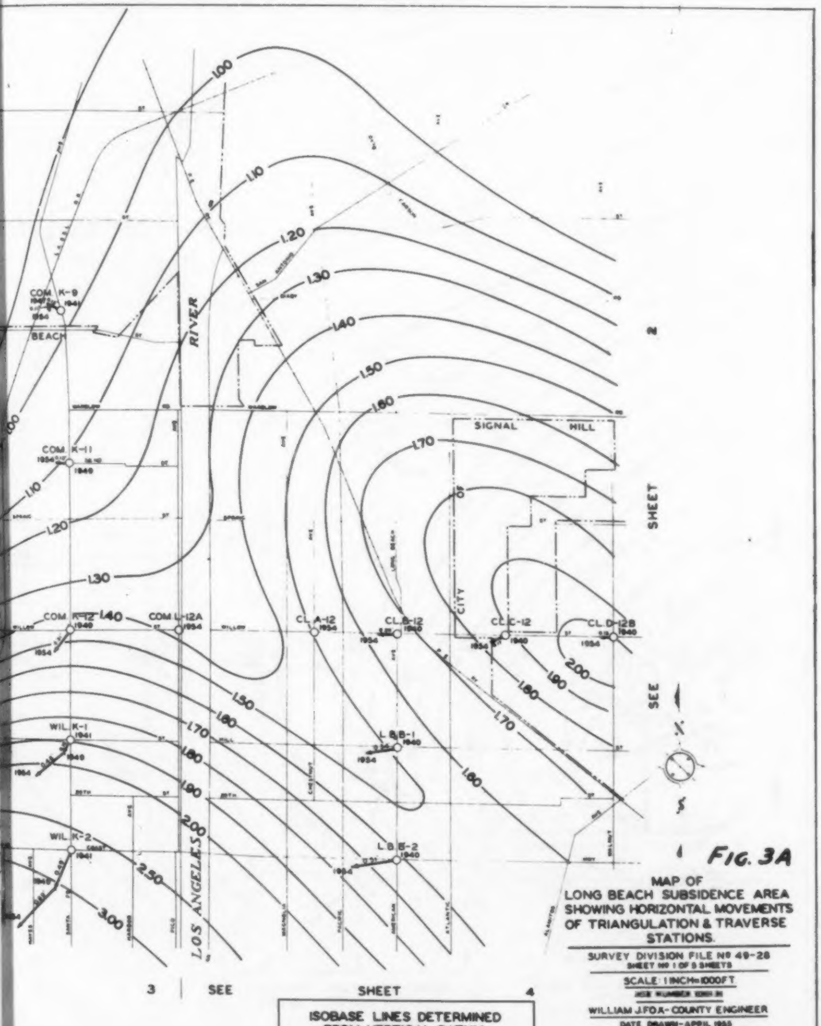
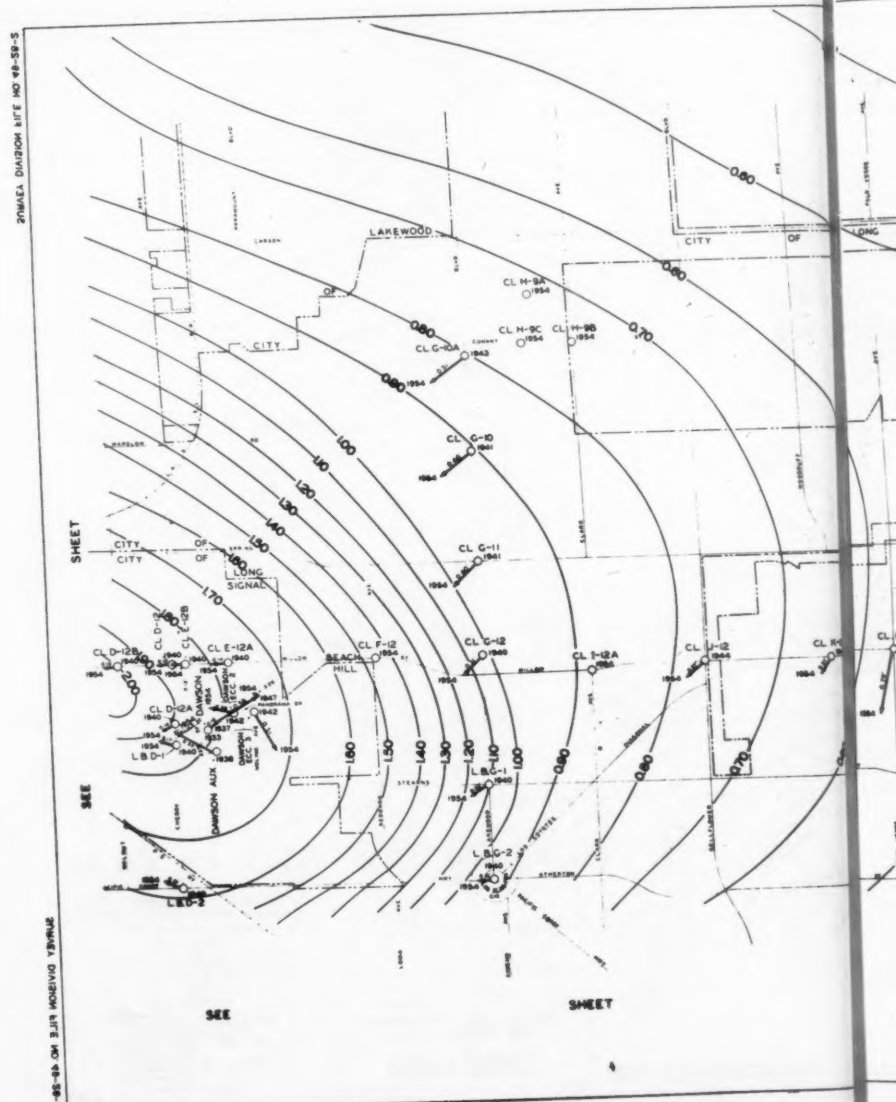


FIG. 3A

MAP OF LONG BEACH SUBSIDENCE AREA SHOWING HORIZONTAL MOVEMENTS OF TRIANGULATION & TRAVERSE STATIONS.


SURVEY DIVISION FILE NO. 49-S8-S
LAKEWOOD
CITY
OF
LONG
SIGNAL
HILL
BEACH
SHEET
SEE
0.60
0.70
0.80
0.90
1.00
1.10
1.20
1.30
1.40
1.50
1.60
1.70
CL H-9A
CL H-9C
CL H-9B
CL G-10A
CL G-10
CL G-11
CL G-12
CL F-12
CL I-12A
CL J-12
CL D-12B
CL D-12A
CL E-12A
L B D-1
L B G-1
L B G-2
L B D-2
DAWSON
DAWSON AUX
DAWSON ECC 3
ATHERTON
STEARNS
CARSON

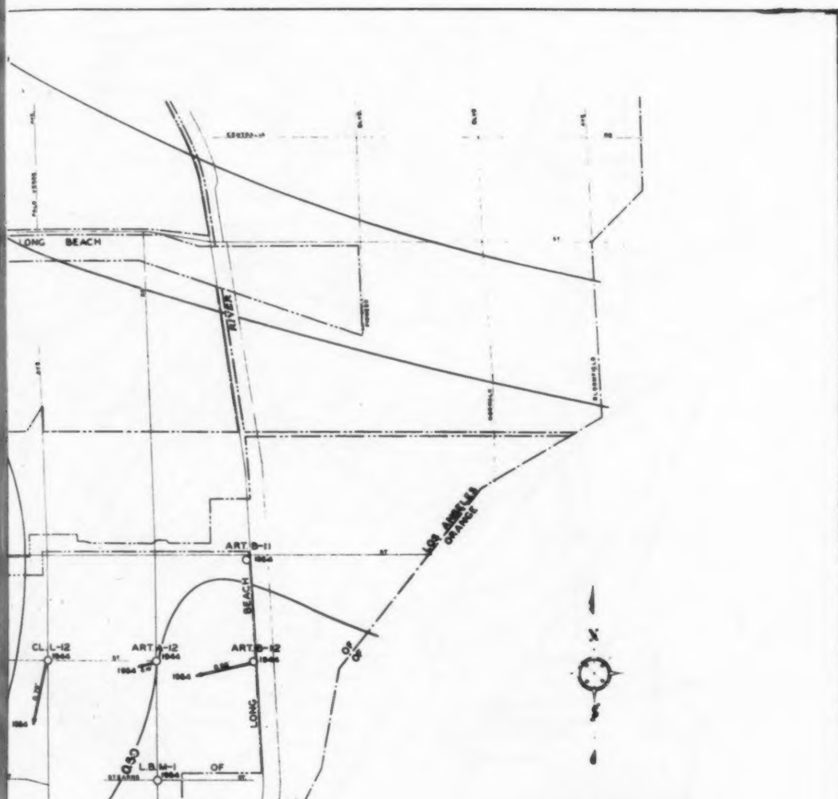
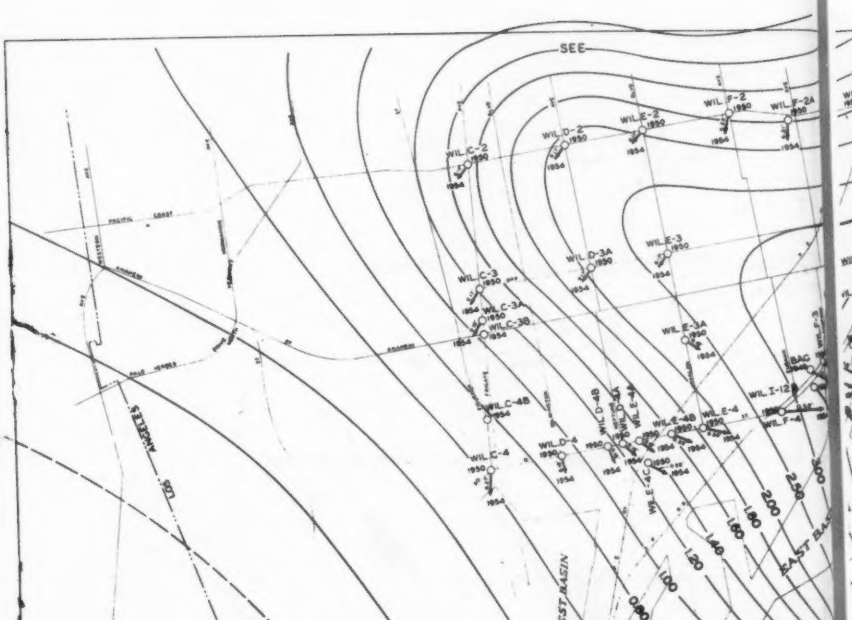
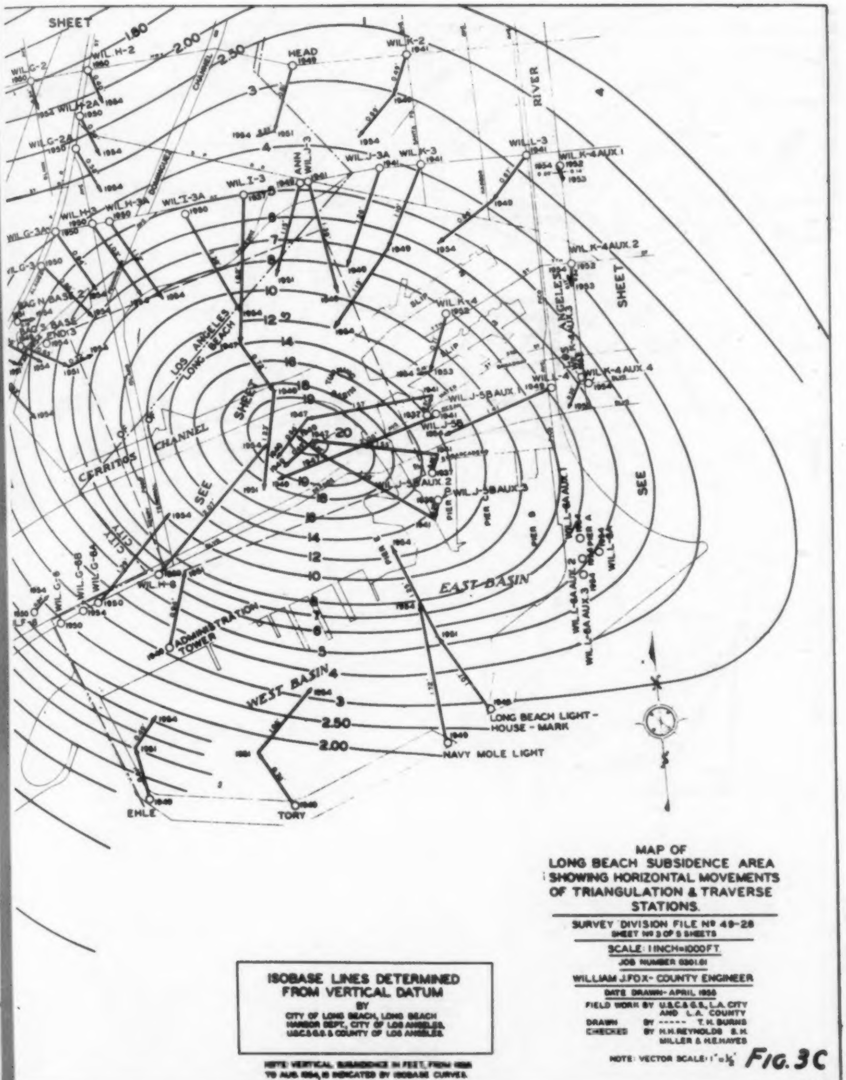
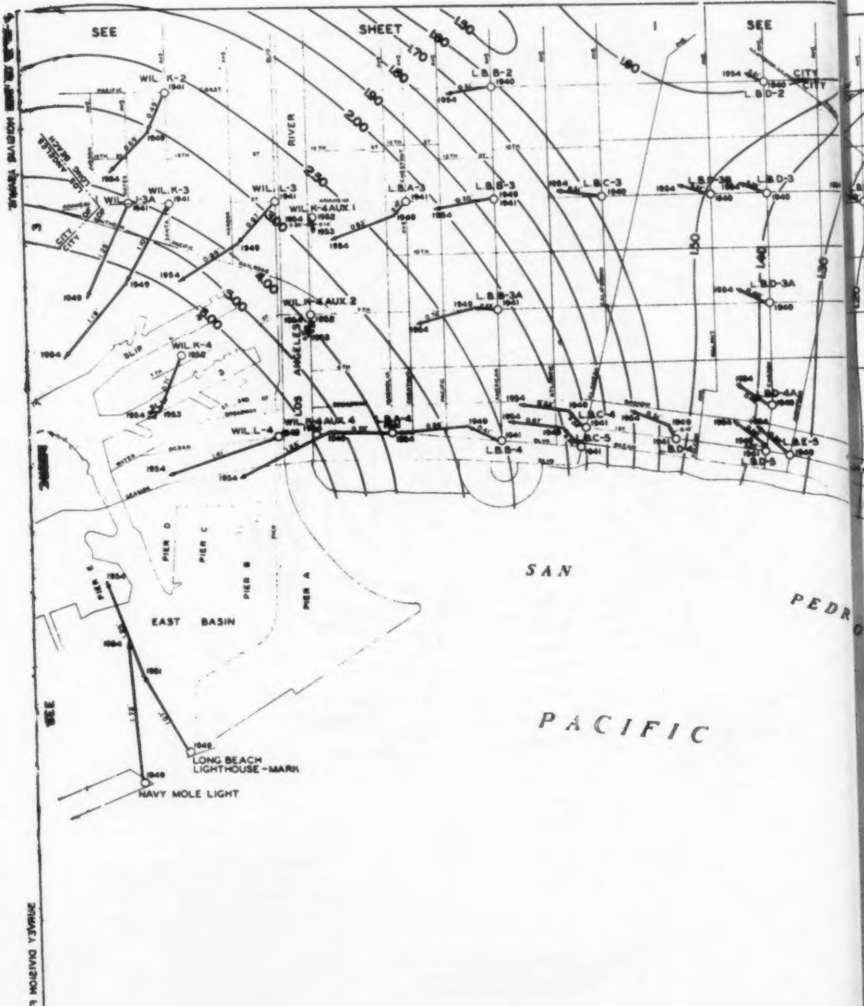

LONG BEACH
OF
RIVER
ART B-11
1964
BEACH
LOS ANGELES
ORANGE
CL. L-12
1964
ART A-12
1964
ART B-12
1964
LONG
0.30
L.B.M-1
1964
OF
CITY
L.B.M-2
1964
COUNTY
COUNTY
SAN GABRIEL
MAP OF
LONG BEACH SUBSIDENCE AREA
SHOWING HORIZONTAL MOVEMENTS
OF TRIANGULATION & TRAVERSE
STATIONS.
SURVEY DIVISION FILE Nº 49-28
SCALE: 1 INCH=1000 FT.
WILLIAM J. FOX-COUNTY ENGINEER
FIELD WORK BY U.S.C.&G.S., L.A. CITY
AND L.A. COUNTY.
DRAWN BY ------ T.H. BURNS
CHECKED BY H.H. REYNOLDS, S.H. MILLER & H.E. HAYES
NOTE: VECTOR SCALE: 1" = 1/2
ISOBASE LINES DETERMINED
FROM VERTICAL DATUM
BY
CITY OF LONG BEACH, LONG BEACH
HARBOR DEPT., CITY OF LOS ANGELES,
NOTE: VERTICAL SUBSIDENCE IN FEET, FROM
TO AUG. 1954, IS INDICATED BY ISOBASE CURVES.

FIG. 3B


SEE
WIL. C-2
WIL. D-2
WIL. E-2
WIL. F-2
WIL. F-2A
WIL. C-3
WIL. C-3A
WIL. C-3B
WIL. D-3A
WIL. E-3
WIL. E-3A
WIL. C-4B
WIL. C-4
WIL. D-4
WIL. D-4B
WIL. E-4B
WIL. E-4
WIL. E-4C
WIL. I-12
WIL. F-4
WIL. E-7A
WIL. E-7
WIL. D-8B
WIL. D-8A
WIL. D-8
AIR
LOS ANGELES
CITY
CITY OF LOS ANGELES
WEST BASIN
EAST BASIN
TURNING BASIN
MAIN CHANNEL
FISH HARBOR
PACIFIC COAST
ANAHEIM
0.00
0.20
0.40
0.60
0.80
1.00
1.20
1.40
1.60
1.80
2.00
2.50
3.00


MAP OF
LONG BEACH SUBSIDENCE AREA
SHOWING HORIZONTAL MOVEMENTS
OF TRIANGULATION & TRAVERSE
STATIONS.
SURVEY DIVISION FILE Nº 49-28
SCALE: 1 INCH=1000 FT.
WILLIAM J. FOX - COUNTY ENGINEER
FIELD WORK BY U.S.C.& G.S., L.A. CITY AND L.A. COUNTY
DRAWN BY T. H. BURNS
ISOBASE LINES DETERMINED
FROM VERTICAL DATUM
BY
CITY OF LONG BEACH, LONG BEACH
HARBOR DEPT., CITY OF LOS ANGELES,
U.S.C.& G.S. & COUNTY OF LOS ANGELES
CERRITOS CHANNEL
EAST BASIN
WEST BASIN
ADMINISTRATION TOWER
LONG BEACH LIGHT-HOUSE-MARK
NAVY MOLE LIGHT
EHLE
TORY
RIVER
FIG. 3C


SEE
SHEET
SEE
SAN
PEDRO
PACIFIC
EAST BASIN
PIER A
PIER B
PIER C
PIER D
LONG BEACH LIGHTHOUSE-MARK
NAVY MOLE LIGHT
WIL. K-2
WIL. K-3
WIL. L-3
WIL. K-4
WIL. L-4
L.B.B-2
L.B.B-3
L.B.B-3A
L.B.B-4
L.B.C-3
L.B.C-4
L.B.C-5
L.B.D-2
L.B.D-3
L.B.D-3A
L.B.A-3
RIVER
LOS ANGELES
SLIP

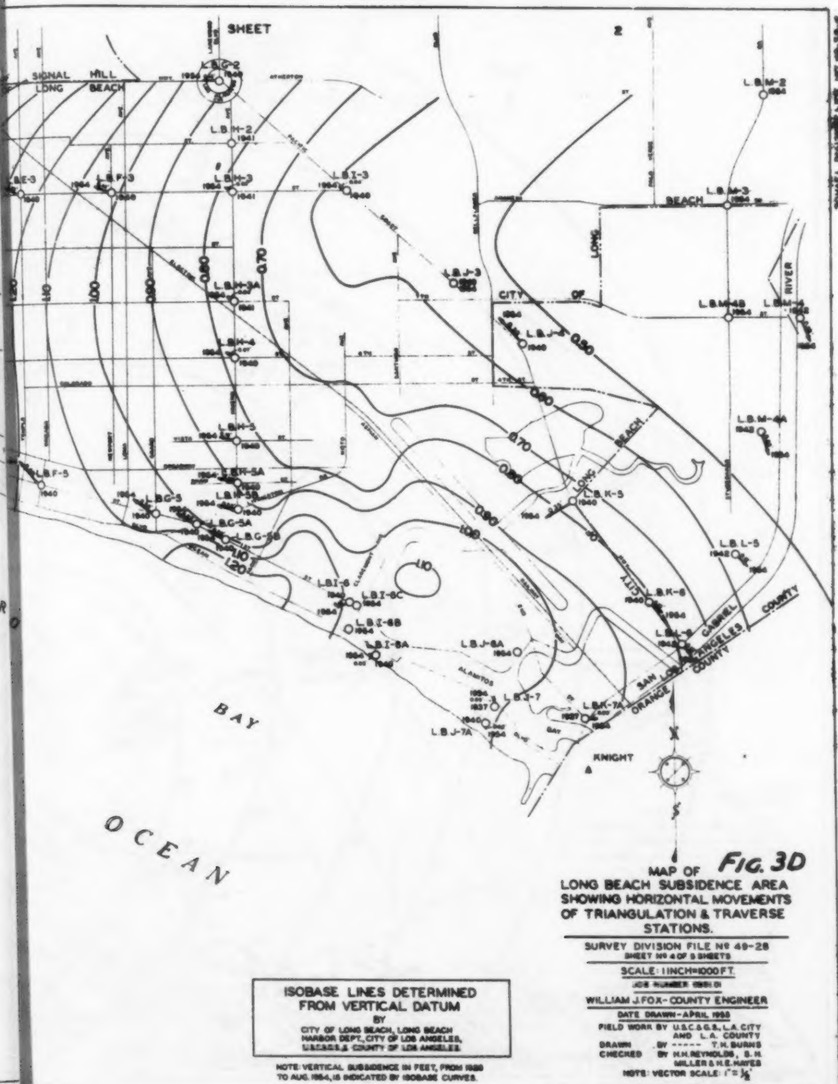
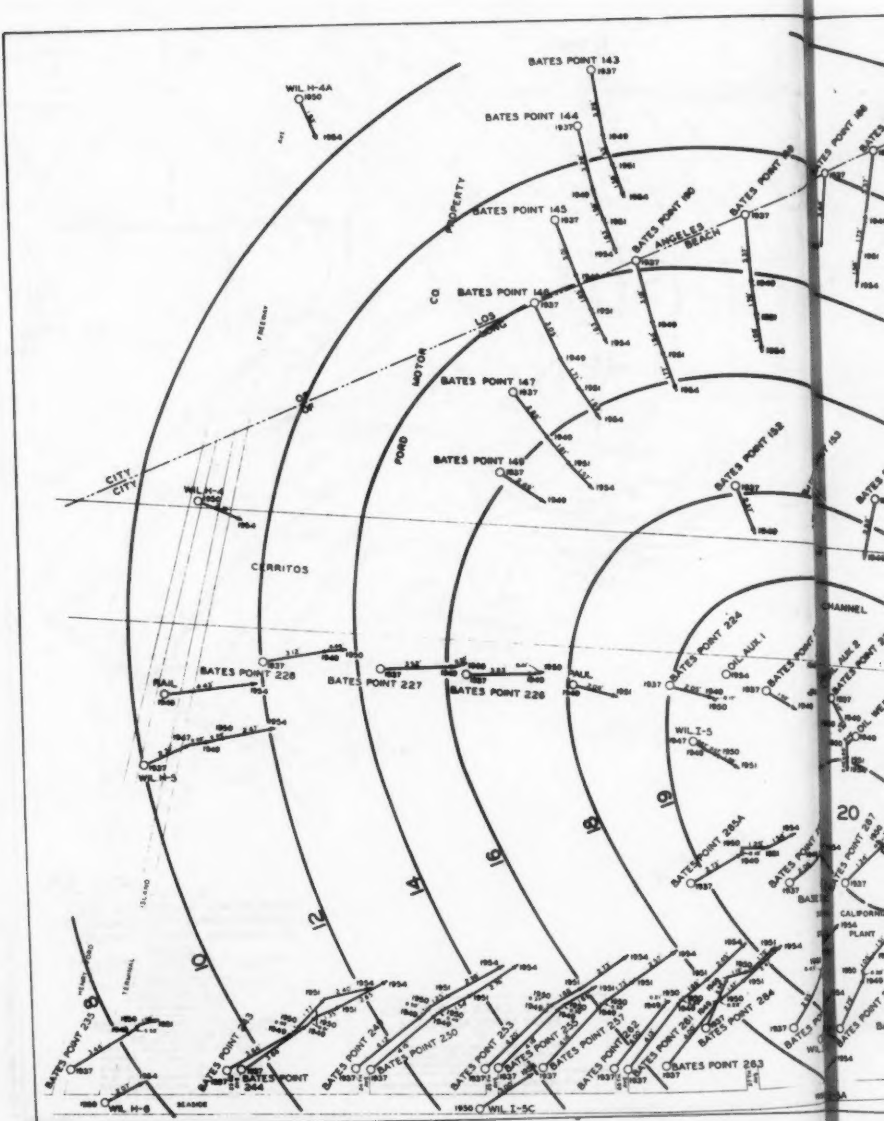
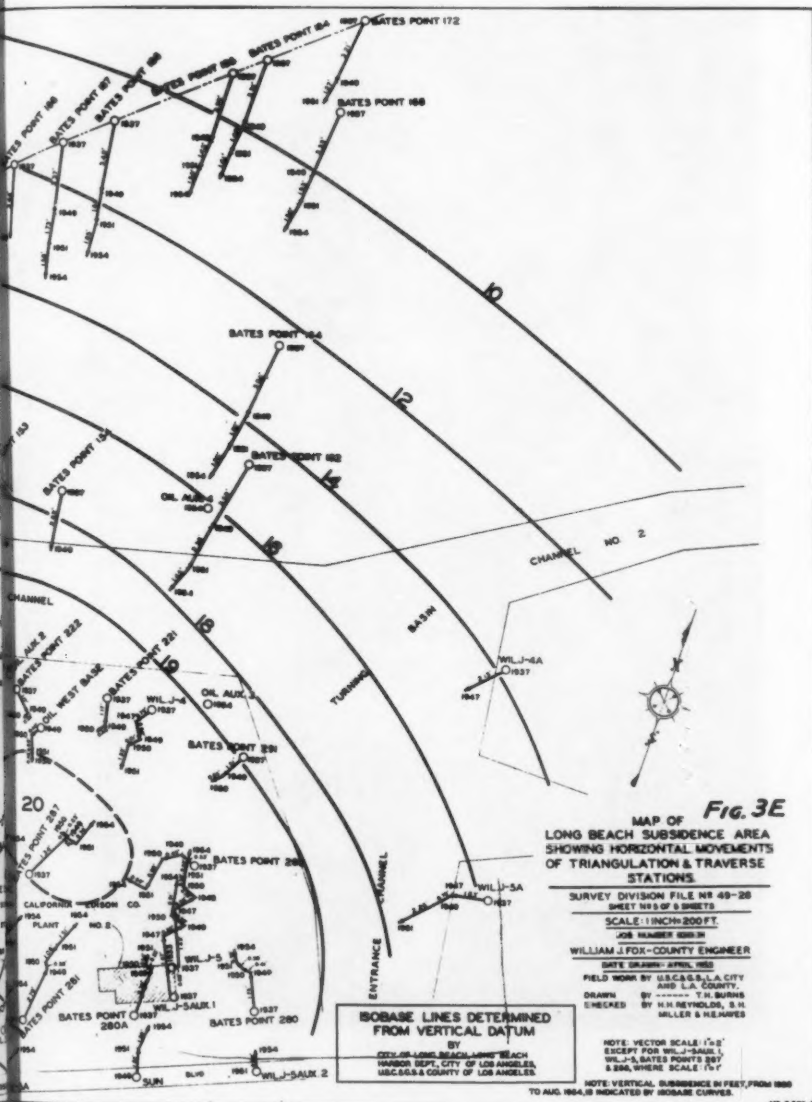
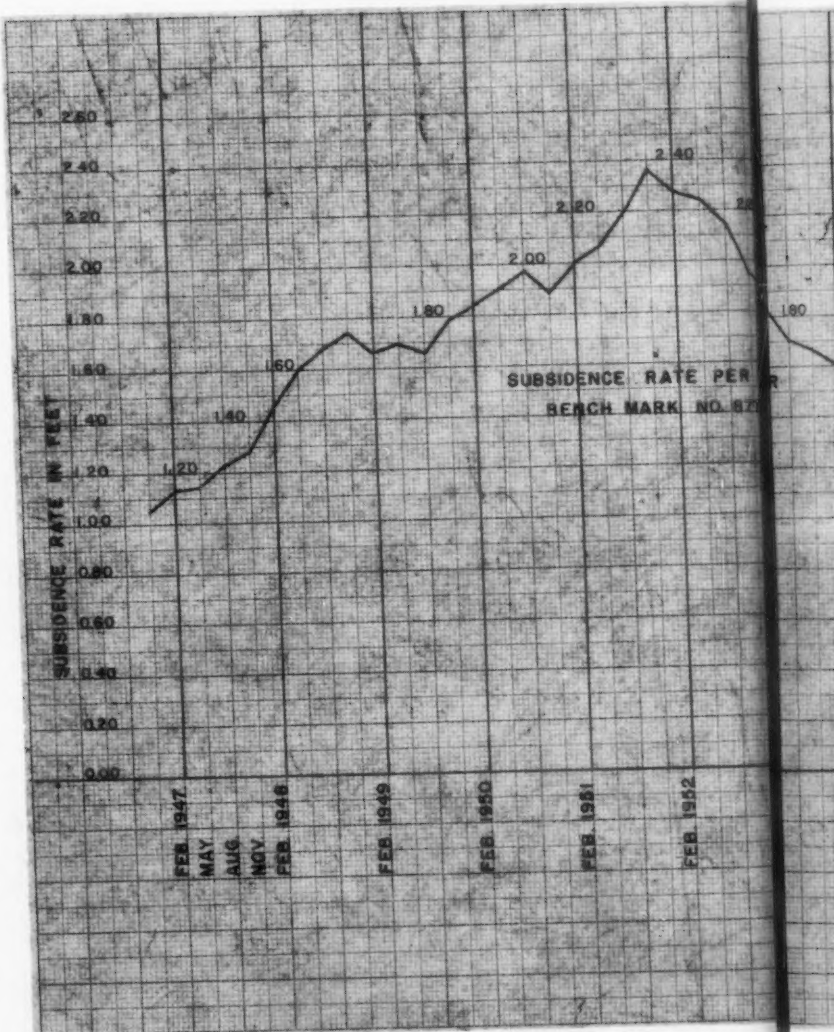


FIG. 3D

MAP OF LONG BEACH SUBSIDENCE AREA SHOWING HORIZONTAL MOVEMENTS OF TRIANGULATION & TRAVERSE STATIONS.


BATES POINT 143
WIL H-4A
BATES POINT 144
BATES POINT 145
BATES POINT 146
BATES POINT 147
BATES POINT 148
PROPERTY
FORD MOTOR CO
LOS ANGELES
LONG BEACH
CITY
CITY
WIL H-4
CERRITOS
BATES POINT 152
CHANNEL
BATES POINT 224
OIL AUX. 1
BATES POINT 226
BATES POINT 227
BATES POINT 228
RAIL
PAUL
WIL H-5
WIL I-5
BATES POINT 285A
BATES POINT 284
BATES POINT 283
BATES POINT 250
BATES POINT 253
BATES POINT 235
WIL H-6
WIL I-5C
CALIFORNIA
PLANT
ISLAND
TERMINAL
8
10
12
14
16
18
19
20


BATES POINT 172
BATES POINT 168
BATES POINT 164
BATES POINT 162
CHANNEL NO. 2
TURNING BASIN
ENTRANCE CHANNEL
CALIFORNIA EDISON CO. PLANT NO. 2
OIL WEST BASE
BATES POINT 221
BATES POINT 222
BATES POINT 281
BATES POINT 287
BATES POINT 280A
BATES POINT 280
WIL.J-4A
WIL.J-5A
WIL.J-5
WIL.J-4
WIL.J-5AUX.1
WIL.J-5AUX. 2
OIL AUX.3
10
12
14
16
18
19
20
ISOBASE LINES DETERMINED FROM VERTICAL DATUM
BY
CITY OF LONG BEACH, LONG BEACH HARBOR DEPT., CITY OF LOS ANGELES, U.S.C.&G.S. & COUNTY OF LOS ANGELES.
MAP OF
LONG BEACH SUBSIDENCE AREA
SHOWING HORIZONTAL MOVEMENTS
OF TRIANGULATION & TRAVERSE STATIONS
SURVEY DIVISION FILE Nº 49-28
SCALE: 1 INCH=200 FT.
WILLIAM J. FOX-COUNTY ENGINEER
FIELD WORK BY U.S.C.&G.S., L.A. CITY AND L.A. COUNTY.
DRAWN BY ------- T.H. BURNS
CHECKED BY H.H. REYNOLDS, S.H. MILLER & H.E. HAYES
NOTE: VECTOR SCALE: 1"=2' EXCEPT FOR WIL.J-5AUX.1, WIL.J-5, BATES POINTS 287 & 280, WHERE SCALE: 1"=1'
NOTE: VERTICAL SUBSIDENCE IN FEET, FROM 1928 TO AUG. 1954, IS INDICATED BY ISOBASE CURVES.
FIG. 3E


SUBSIDENCE RATE IN FEET
2.60
2.40
2.20
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
1.20
1.40
1.60
1.80
2.00
2.20
2.40
SUBSIDENCE RATE PER
BENCH MARK NO.
FEB. 1947
MAY
AUG.
NOV.
FEB. 1948
FEB. 1949
FEB. 1950
FEB. 1951
FEB. 1952

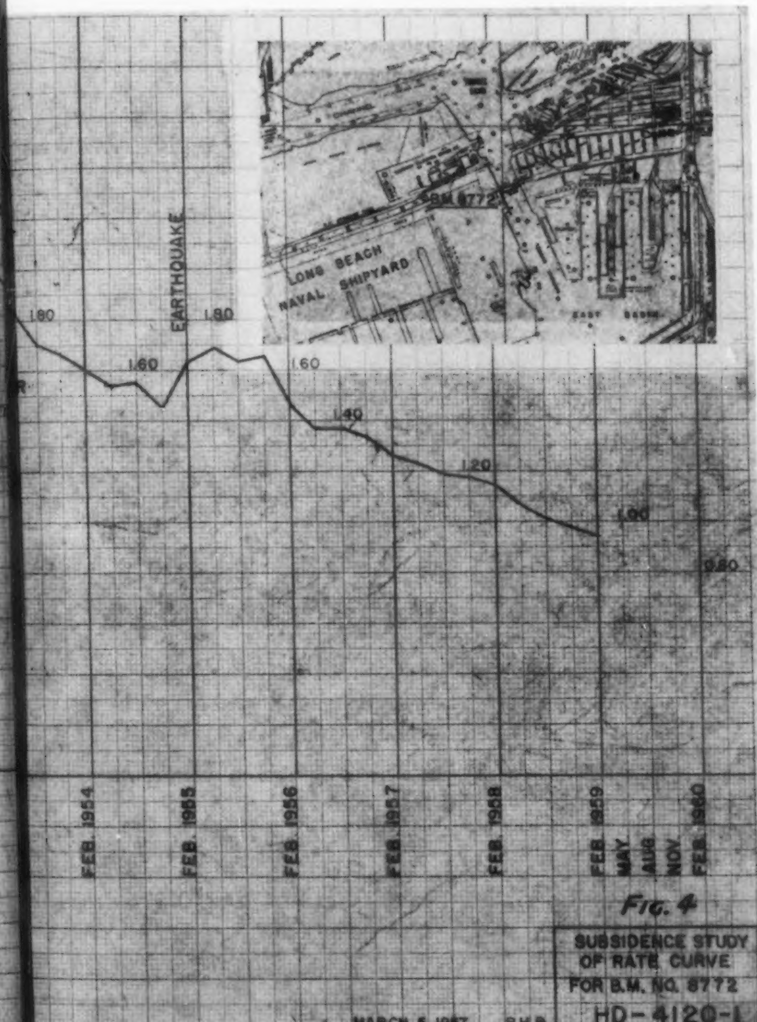
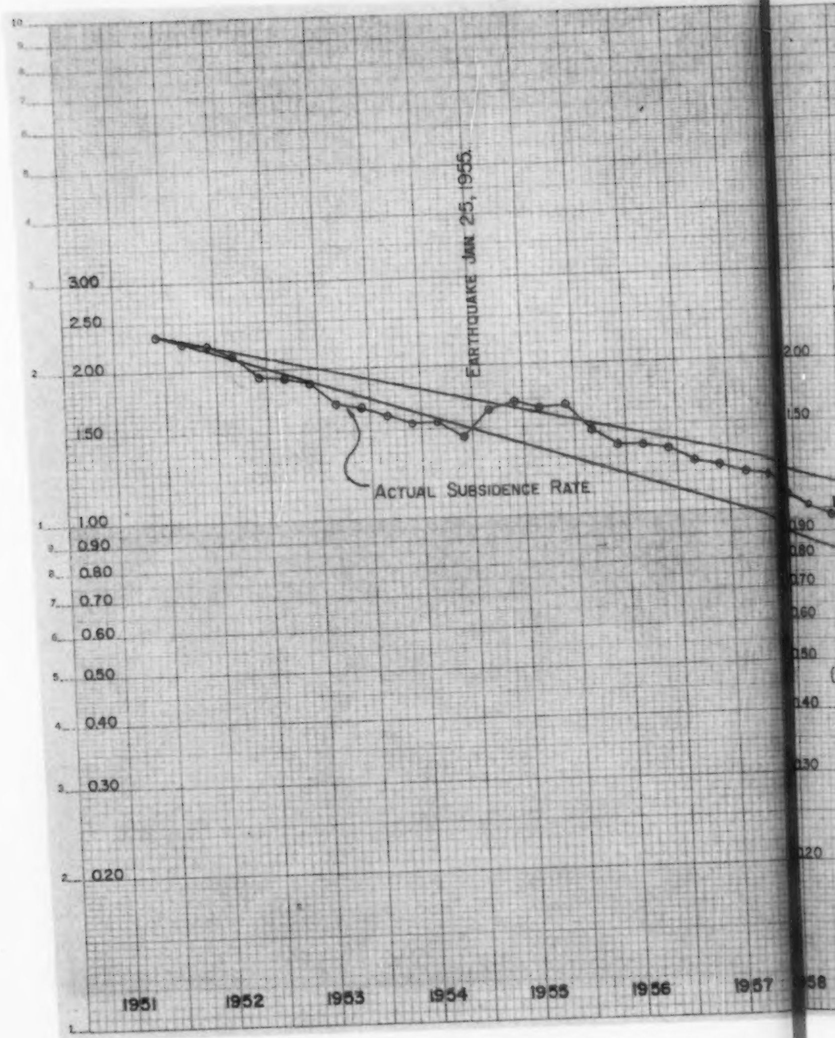
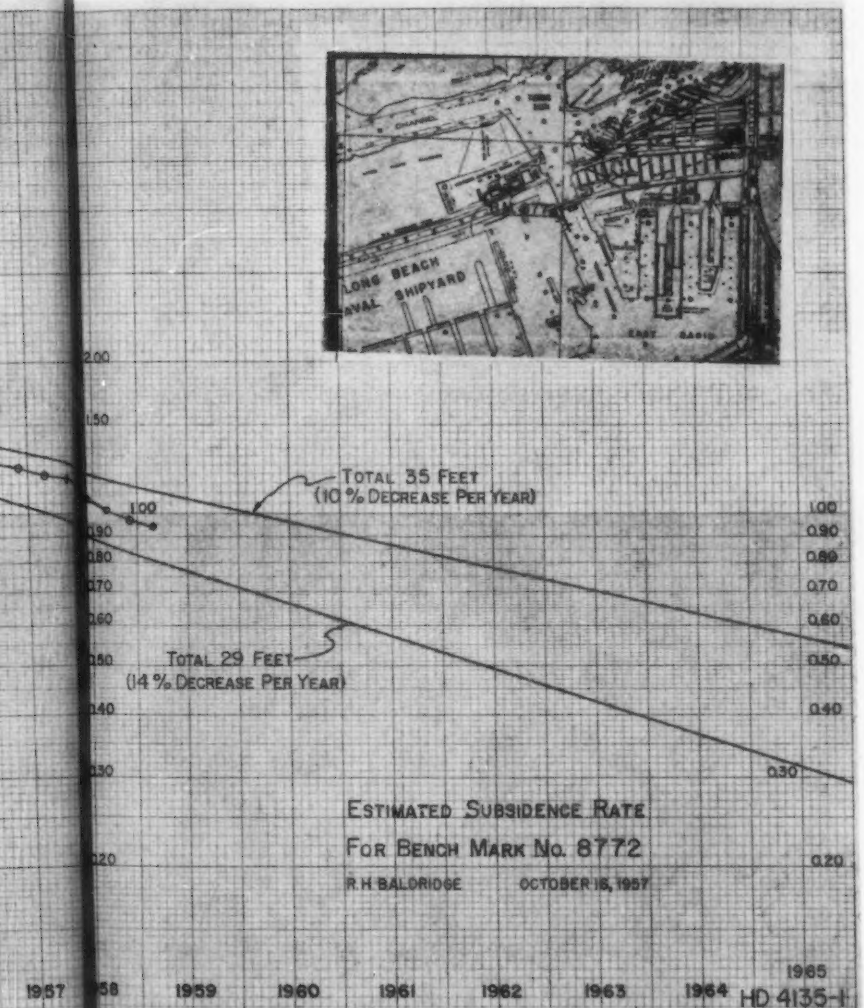
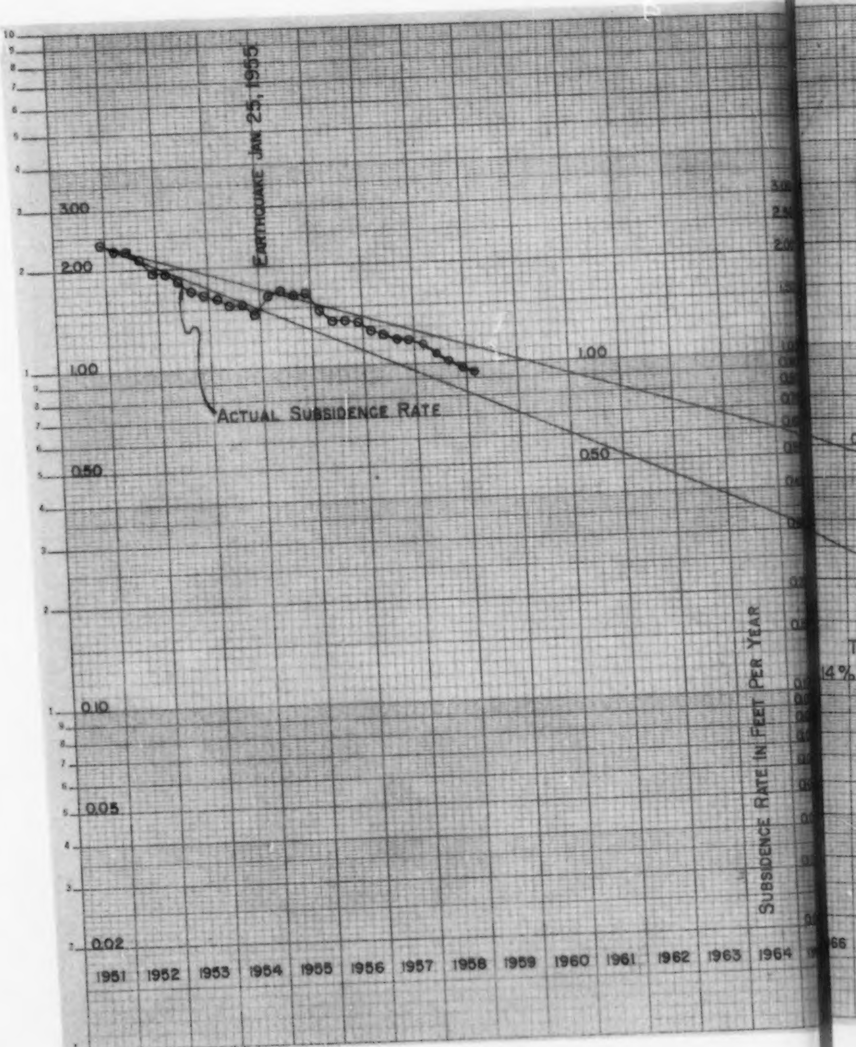
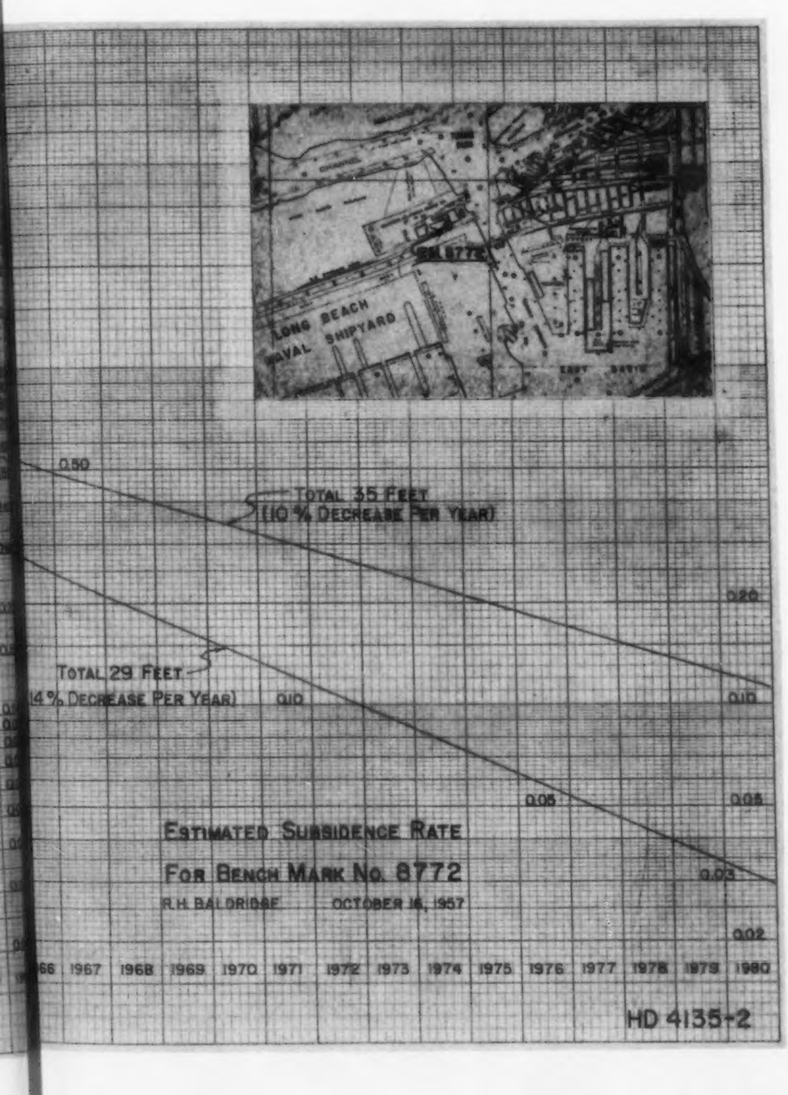

LONG BEACH
NAVAL SHIPYARD
BM 8772
EARTHQUAKE
1.80
1.80
1.60
1.60
1.40
1.20
1.00
0.80
FEB. 1954
FEB. 1955
FEB. 1956
FEB. 1957
FEB. 1958
FEB. 1959
MAY
AUG
NOV
FEB. 1960
SUBSIDENCE STUDY
OF RATE CURVE
FOR B.M. NO. 8772
HD-4120-1
MARCH 5, 1957
R.H.B.

FIG. 4

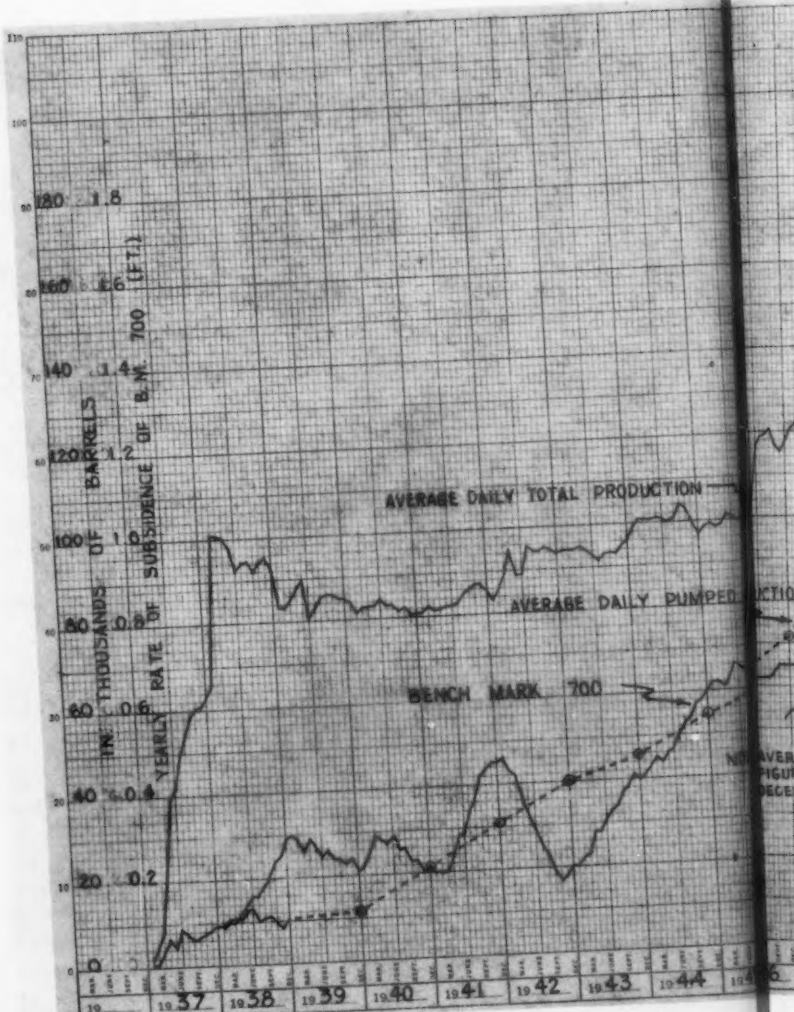

EARTHQUAKE JAN 25, 1955.
ACTUAL SUBSIDENCE RATE
3.00
2.50
2.00
1.50
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
1951
1952
1953
1954
1955
1956
1957

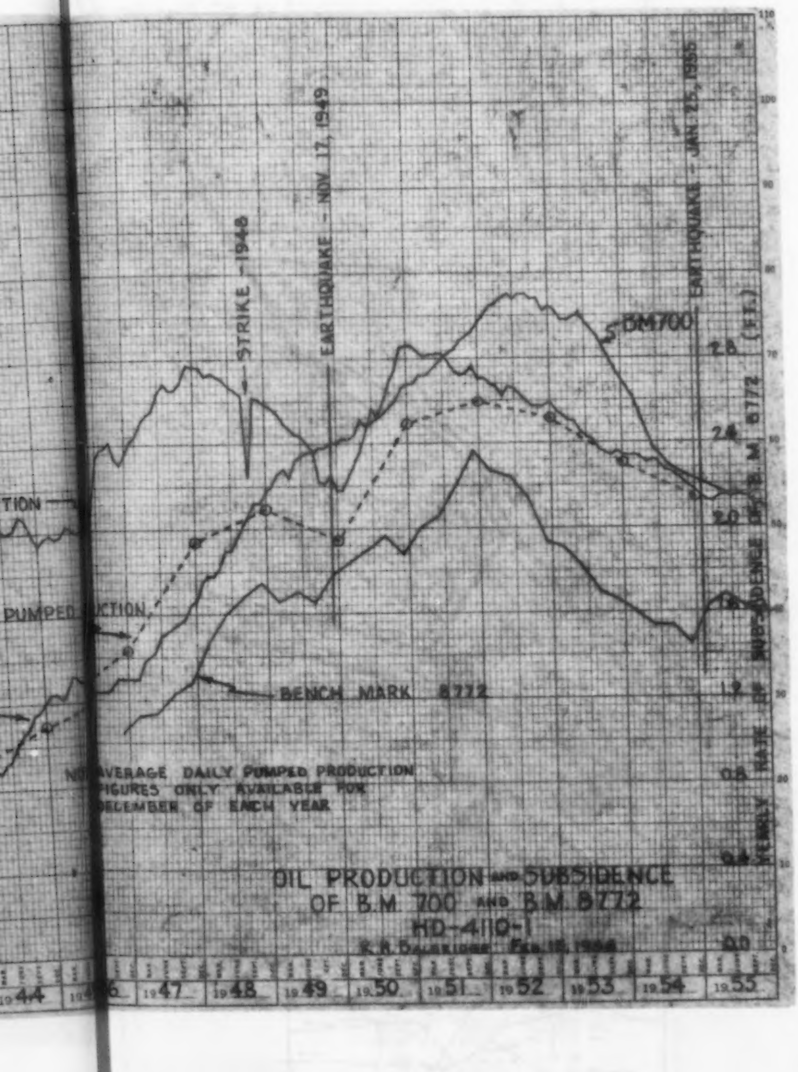

LONG BEACH
NAVAL SHIPYARD
TOTAL 35 FEET
(10 % DECREASE PER YEAR)
TOTAL 29 FEET
(14 % DECREASE PER YEAR)
ESTIMATED SUBSIDENCE RATE
FOR BENCH MARK No. 8772
R.H. BALDRIDGE OCTOBER 16, 1957
2.00
1.50
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
1957
1959
1960
1961
1962
1963
1964
1965
HD 4135-1


EARTHQUAKE JAN 25, 1955
ACTUAL SUBSIDENCE RATE
SUBSIDENCE RATE IN FEET PER YEAR
3.00
2.00
1.00
0.50
0.10
0.05
0.02
1.00
0.50
1951
1952
1953
1954
1955
1956
1957
1958
1959
1960
1961
1962
1963
1964

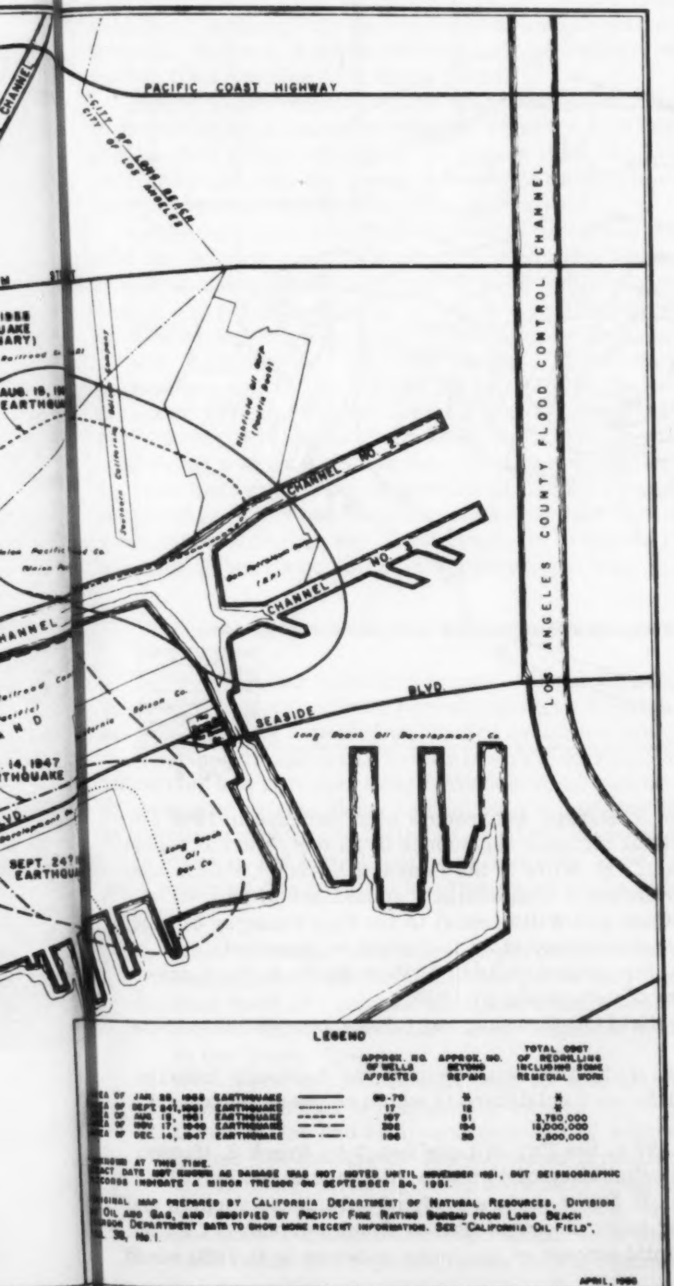


LONG BEACH
NAVAL SHIPYARD
BM 8772
TOTAL 35 FEET
(10 % DECREASE PER YEAR)
TOTAL 29 FEET
(4 % DECREASE PER YEAR)
SUBSIDENCE RATE IN FEET PER YEAR
0.50
0.20
0.10
0.10
0.10
0.05
0.05
0.03
0.02
1967
1968
1969
1970
1971
1972
1973
1974
1975
1976
1977
1978
1979
1980
ESTIMATED SUBSIDENCE RATE
FOR BENCH MARK No. 8772
R.H. BALDRIDGE
OCTOBER 16, 1957
HD 4135-2


AVERAGE DAILY TOTAL PRODUCTION
AVERAGE DAILY PUMPED
BENCH MARK 700
IN THOUSANDS OF BARRELS
YEARLY RATE OF SUBSIDENCE OF B.M. 700 (FT.)
180 1.8
160 1.6
140 1.4
120 1.2
100 1.0
80 0.8
60 0.6
40 0.4
20 0.2
0
1937
1938
1939
1940
1941
1942
1943
1944


STRIKE -1948
EARTHQUAKE - NOV 17, 1949
EARTHQUAKE - JAN 25,
BM700
BENCH MARK 8772
AVERAGE DAILY PUMPED PRODUCTION
FIGURES ONLY AVAILABLE FOR
DECEMBER OF EACH YEAR
OIL PRODUCTION AND SUBSIDENCE
OF B.M 700 AND B.M. 8772
HD-4110-1
2.8
2.4
2.0
1.6
1.2
0.8
0.4
0.0
1944
1947
1948
1949
1950
1951
1952
1953
1954
1955


MAP OF
WILMINGTON OIL FIELD
LOS ANGELES COUNTY, CALIF.
Scale
U.S. ALT. 101
DOMINGUEZ CHANNEL
ANAHEIM
JAN. 25, 1955
EARTHQUAKE
(PRELIMINARY)
HENRY FORD AVE.
CERRITOS CHANNEL
NOV. 17, 1949
EARTHQUAKE
DEC. 14, 1947
EARTHQUAKE
ISLAND
TERMINAL
BLVD.
U.S. NAVY
VICINITY
MAP
Area
Shown
SCALE


PACIFIC COAST HIGHWAY
CITY OF LONG BEACH
CITY OF LOS ANGELES
LOS ANGELES COUNTY FLOOD CONTROL CHANNEL
CHANNEL NO. 2
CHANNEL NO. 3
SEASIDE BLVD.
Long Beach Oil Development Co.
Long Beach Oil Dev. Co.
LEGEND
APPROX. NO. OF WELLS AFFECTED
APPROX. NO. BEYOND REPAIR
TOTAL COST OF REDRILLING INCLUDING SOME REMEDIAL WORK
3,750,000
2,800,000

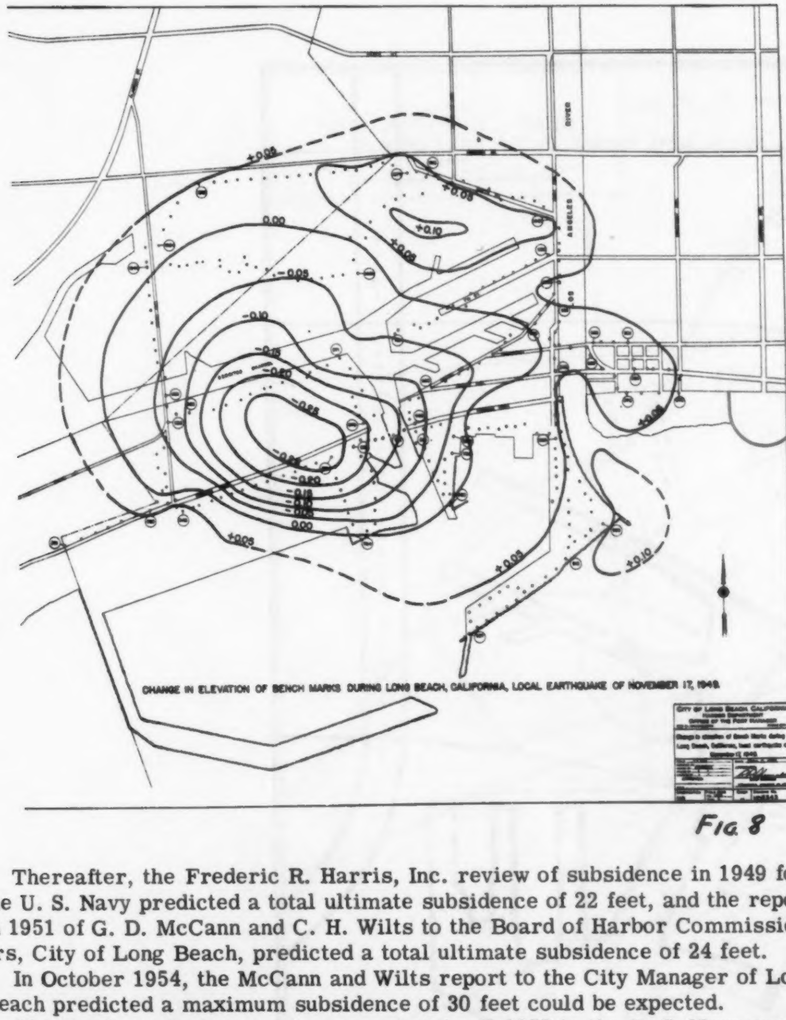


Fig. 8

Thereafter, the Frederic R. Harris, Inc. review of subsidence in 1949 for the U. S. Navy predicted a total ultimate subsidence of 22 feet, and the report in 1951 of G. D. McCann and C. H. Wilts to the Board of Harbor Commissioners, City of Long Beach, predicted a total ultimate subsidence of 24 feet.

In October 1954, the McCann and Wilts report to the City Manager of Long Beach predicted a maximum subsidence of 30 feet could be expected.

The Frederic R. Harris, Inc. report of April 1955 to the U. S. Navy predicted approximately 30 feet of subsidence by 1970.

A 1955 report to the Richfield Oil Company indicates a possible maximum of 35 feet.

In January 1956, Frank S. Hudson made a report to the Assembly Interim Committee of Judiciary California Legislature in which he predicted a total of 54 feet.

A report in September 1957 to the City of Long Beach by Frank S. Hudson estimated a total maximum subsidence of 43 feet would be attained by 1977.

In December 1957, Dr. U. S. Grant made a study of subsidence reports and predictions and in his report to the Board of Harbor Commissioners, City of Long Beach, estimated the total amount of maximum subsidence in 1980 would be 34 feet.

Many methods have been used in trying to determine the values in the above predictions. All are based on generally accepted theories, but each must depend upon one or more factors that cannot be precisely measured or determined. Although the underground measurements are to some extent conjectural, they are of sufficient accuracy for reliable subsidence predictions and oil field planning over many years.

Surface measurements can be made with a very low percentage of error. In August 1958, a maximum subsidence at the easterly end of Terminal Island had reached a total of 25 feet. In August 1954, the Los Angeles County Engineer rechecked the horizontal movement in this same area and found a maximum movement of nine feet from 1937 to 1954.

As time goes on, the percentage of error of the factors necessary to compute the ultimate amount of subsidence is being reduced. Present estimates of a total ultimate subsidence of 28 to 35 feet are based on the best information obtainable today, but it may be these figures will have to be revised when more sensitive methods of underground measurement are devised.

With the exception of the 1954 McCann-Wilts report, the principal investigators have now concluded that the subsidence figures estimated by them for the years 1970 to 1972 are not final but that more settlement of the land surface may be expected thereafter, probably at a considerably reduced rate.

Pending a more accurate determination of the "lag" period and a more specific finding as to the mechanics of the forces responsible for the "lag", engineering policy has been revised in the direction of expecting greater ultimate subsidence than any of the currently available reports would indicate, and making provision therefor wherever possible.

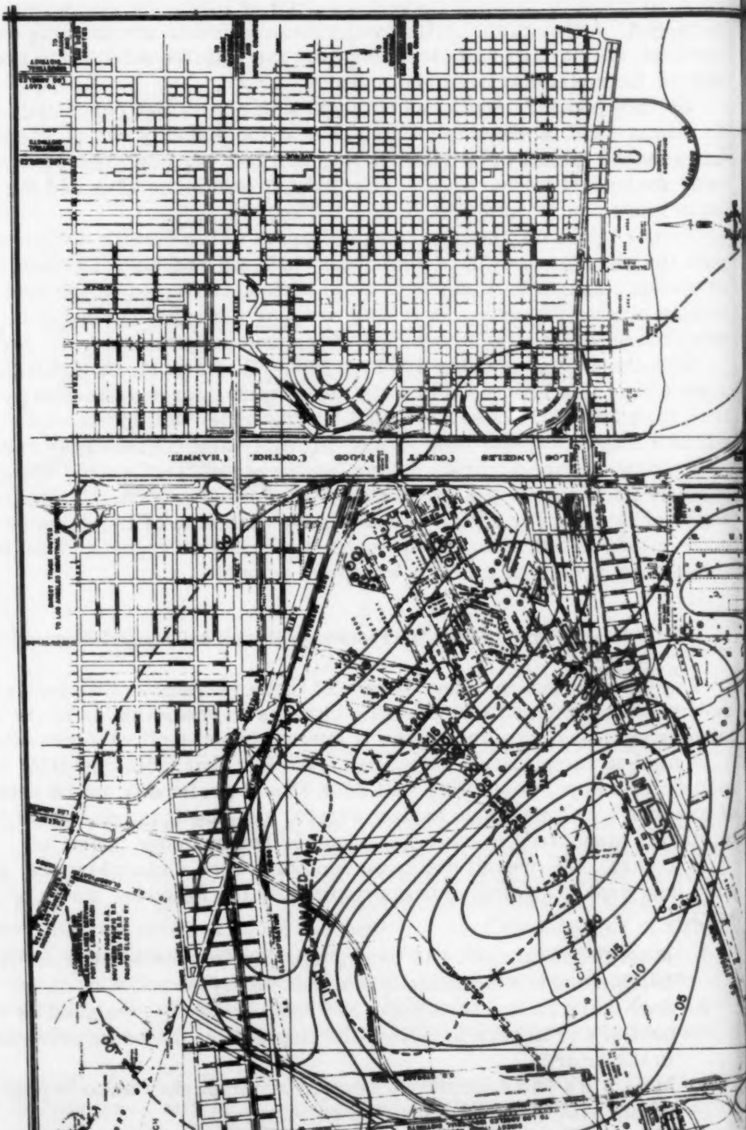
Influence of Subsidence on Planning, Construction and Expenditures

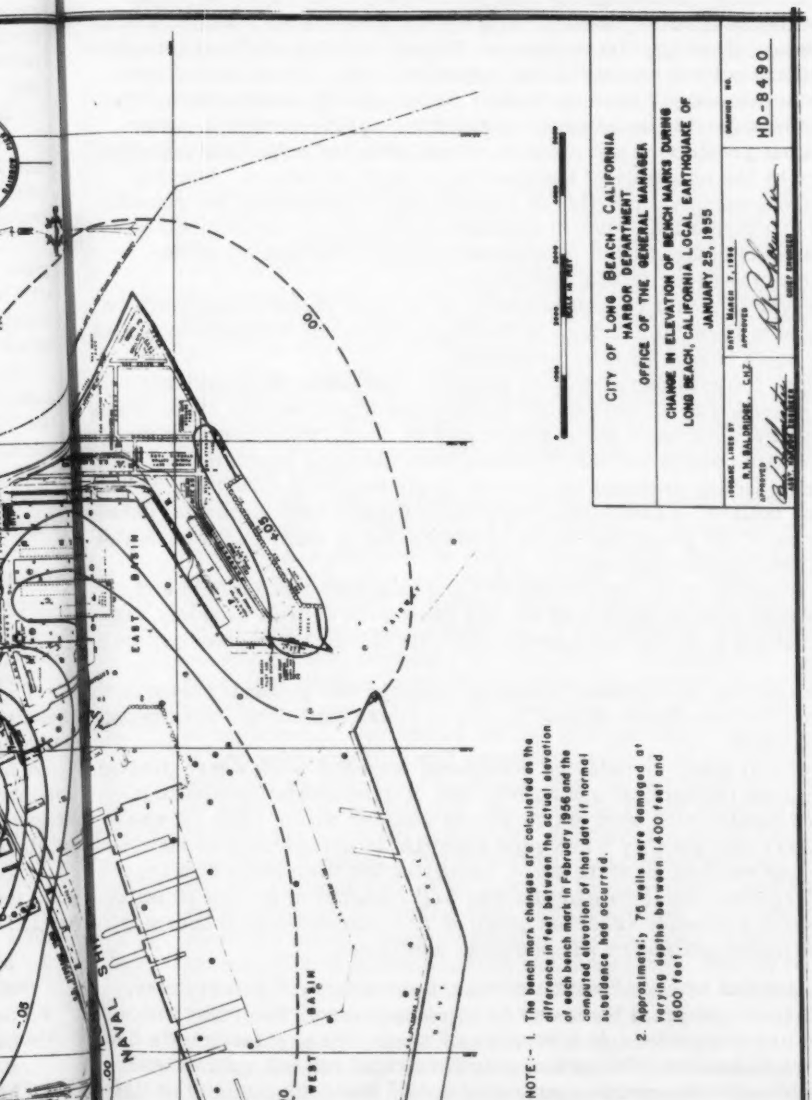
The variable but continued nature of these vertical and horizontal movements, combined with periodic changes in prediction as to the maximum movements to be expected and their timing, has complicated remedial work.

A further complication arises from the fact that most operating waterfront industries and port operating facilities should preferably not be established at excessive heights above harbor waters, and that it is not always practical to elevate structures to their full required height in one operation.

In general, the overall policy on remedial work dictated by past events and good judgment has resulted in approximately the following status of corrective works:

1. Most remedial work had been purposely delayed or only partially accomplished if economically and practically possible to do so.
2. Such work as had been done, had usually been so designed as to be a part of and to permit further development of the same protective work in the future.
3. Most work had necessarily been so accomplished as to permit coordination with that of neighboring owners.
4. Coordination had been accomplished by simple processes. Construction permits in the Harbor District clear through the Port of Long Beach Engineering Division, if the work is in Long Beach. Thus, it is usually possible to determine the plans of others. In addition, five special subsidence committees, representative of the principal land owners of each of the five separate areas having common flooding problems, have


LOS ANGELES COUNTY FLOOD CONTROL CHANNEL
CHANNEL
CERRITOS
VICINITY SKETCH
U. S. NAVAL


CITY OF LONG BEACH, CALIFORNIA
HARBOR DEPARTMENT
OFFICE OF THE GENERAL MANAGER
CHANGE IN ELEVATION OF BENCH MARKS DURING
LONG BEACH, CALIFORNIA LOCAL EARTHQUAKE OF
JANUARY 25, 1955
HD-8490
NOTE:- 1. The bench mark changes are calculated as the difference in feet between the actual elevation of each bench mark in February 1956 and the computed elevation of that date if normal subsidence had occurred.
2. Approximately 75 wells were damaged at varying depths between 1400 feet and 1600 feet.
U. S. NAVAL STATION
NAVAL
WEST BASIN
EAST BASIN
.00
-.05
+.05

been sponsored by the Port Engineering Division. These engineering committees meet periodically—usually monthly on days of extreme high tide—to study, discuss and report on the state of levee development. Questions of relative responsibility for work have been subordinated to the principal policy of cooperation. The engineering staffs and managements are quickly advised of any apprehensions of these committees.

5. Many areas nearing the hazard point have been left uncorrected. This policy was fixed in some cases by the desire of the owners to obtain the latest predictions and advices before initiating rehabilitation and in others by the necessity of maintaining current operations. Internal revenue department policies of questioning tax deductions for remedial work in anticipation of future subsidence, and of privately-owned improvements being injured by subsidence, have also delayed or determined the order of some activities.
6. Some type of actual rehabilitation in the form of partial, temporary or permanent physical barriers to flooding had already been accomplished along some twenty miles of waterfront.
7. Roughly, 100 millions of dollars properly chargeable to subsidence had been expended by all parties for surface operations by late 1958. These expenditures had been borne principally by about twelve large public and private owners and their lessees, with the Long Beach Harbor Department having assumed the largest single burden at upward of 33 million dollars. Lesser ownerships have usually been incidental beneficiaries of the protective works of others, but in many cases have also expended considerable sums.
8. Most remedial work has necessarily had an element of addition and betterment incorporated, and for this reason the overall economy and utility of the area has continued to increase during the reconstruction period.
9. The magnitude of individual remedial projects was generally being increased because better defined trends of future subsidence were becoming available.
10. The general policy of delaying permanent remedial work where practicable has its limitations, and activity had started ascending toward a peak of construction work which will be attained during 1959. It was necessary that the City's remedial program be accelerated as subsidence had reached an acute stage, impairing the usefulness of many Port facilities; immediate action was well considered in view of reliable opinions recently rendered based on accumulated data concerning both ultimate subsidence and remedial methods.

Every instance of remedial work represents some form of compromise, but with the overall program there can be no compromise. The many miles of waterfront are compacted into a very small area. The greatest single flow of flood waters in Southern California, with very rapid run-off, passes with high velocity through this compact subsiding area. Since the capacity of this flood control channel is largely influenced by ocean level at its mouth, radical changes were necessary in the levees and bridge structures along the lower reaches of this stream.

An incidental benefit of the required removal of bridge structures which had become serious flood hazards was that there was no practicable method of replacement of the bridges which would not vastly improve the operations of

the Harbor District because of the railroad and highway grade separations that would be created. This would greatly improve the traffic situation.

Also, because of the physical changes brought about by the undertaking of bridge and access remedial work, it is necessary to acquire some 100 acres of privately-owned and developed property which will be damaged or occupied by these works. A limited amount of this acquisition will eventually further serve the functioning of the Port, aside from meeting the access needs.

A few of the many different types of remedial work required in the subsiding area are described as follows:

1. Simple truck-dumped earth fill dikes are constructed where danger of flooding is momentary (at periods of extreme high tide or surge) and back area conditions justify. The same procedure applies for temporary installations awaiting more permanent works.
2. Elaborately designed and consolidated earth levees with center cores of impermeable muds or clays are indicated and have been used where large land areas are used exclusively for oil production; where there is little or no habitation and where there are no waterfront industrial or port facilities. Such installations require extensive drainage and pumping facilities and/or dewatering systems to supplement the levees and present a steadily increasing pumping and drainage program within the protected areas.
3. Large scale filling of low areas to elevate them to heights calculated to remain above the elevation of high tide or of ground water are often used. In a comparatively virgin area, free of surface structures, this procedure is simple and ideal; however, there are few such areas. Where the intensity of surface dollar investment is high, this procedure either becomes less feasible, or, if indicated as ultimately essential, it becomes a slow process requiring years to accomplish. In the latter case, it is usually found that with the exception of oil wells and their accessories and a limited few buildings, all surface improvements must be abandoned. Because of the fact that future ground water table levels in presently dry fills will permeate new fills as the area subsides, essentially all abandoned facilities must be removed or rendered harmless by some positive step, i.e., the plugging of abandoned utility lines.
4. Raising the elevation of valuable structures poses the question of economic justification. Due to rapidly changing technology and relative cost, it is usually more economical to rehabilitate with an entirely different form of construction; and fortunately, this new form of construction usually permits greater load carrying capacity, greater safety, elimination of fire hazards and longer life. Not the least of the reasons for these results is the necessity for so designing the structure that it or its surroundings can be raised in the future or that it can be subjected to increased loads in the future.
5. Horizontal and vertical land movements causing sinkage, tilting and rotation of supports for various bridges in the area has necessitated remedial work ranging from periodic stress relieving measures on some, to raising and reconstruction of others.
6. Storm drains serving the Harbor Area have in some cases completely reversed direction of flow. Some storm drains become sources of ocean flooding and must be abandoned. During the interim periods, tidewater is sometimes diverted through manholes and street gutters to

a different gravity drainage system. In early stages, Calco gates were effective at the ocean ends.

7. Sewers are alternately crushed or parted at the joints. The most important storm drain in the area is an abandoned 48-inch sanitary sewer which receives most of the ground water flow through its factured laterals and conveys it to a pumping station originally constructed to handle sewage. In isolated cases, sewage is now handled by septic tank. Sewers serving important structures are maintained.
8. Steel and cast iron pipelines generally fail from time to time by reason of the induced stresses from earth movements. In new installations, as well as in repairs, the maximum flexibility is designed into the lines. Breaks may be tensile failures, buckling or marked horizontal offsets which are usually bridged by flexible couplings or contractable connections. These failures can rarely be predicted as to exact position.
9. Railroad tracks have buckled even though designed for normal expansion. Relief can be quickly provided by cutting the buckled section.
10. Extremely massive underground reinforced concrete conduits have failed. Replacements are above ground where practicable.
11. Major buildings, well equipped with contraction joints for normal conditions, have required adjustment to meet horizontal movements. One building structure, 120 feet by 832 feet, shortened 1.06 feet on one side and a lesser degree on the other. Remedial work followed failure of the weakest panel in the building, and took the form of cutting a strip from the building for its full width and partially replumbing the two halves before reconnecting the severed parts. Early 1955 brought more shortening to this same building. New structures are designed to accommodate these movements.
12. Where possible, large buildings which have been subjected to unusual lateral forces will not be repaired or remedied until the structures are elevated. Buildings are watched with care to observe possible sources of hazard.
13. Street grades are often seriously affected, causing drainage problems. As ground water tables approach subgrade, which they do despite use of pumping systems, streets and tracks are adversely affected.
14. Basically, any combination of structures in contact with the earth which have radically different elastic qualities will be differently affected. For example, a concrete gutter will be stressed to the point of tilting manholes or catch basins, whereas the asphaltic concrete adjacent will flow around obstructions.
15. Obviously, many forms of continuous structures are presently impracticable in the area and the principles of continuity must be carefully applied.

This list of types of remedial work is not complete, but indicates the hundreds of individual projects involved. The work requires coordination, which is effected largely through the Engineering Division of the Port. Coordination of a much higher degree will be involved in the future. In some instances, the diversity of ownerships indicates that coordination may not be feasible within the allotted time limits even for surface effects. In such cases the principal agency involved, usually the Harbor Department, may have to acquire the property necessary for completion of the protective works.

Influence of Subsidence on Operations

The waterfront remedial works which must serve to rehabilitate the operating Port facilities must also exclude high tides for many years until indicated land raising operations are completed. These waterfront facilities cannot function without the adequate rail and highway approaches which are elements of the Port. These approaches must be raised and rerouted, thus disturbing all adjacent lands. This work must necessarily be so accomplished as to permit coordination with that of neighboring owners.

This land raising program requires close coordination in order to maintain all important operations at all times. Therefore, even the final solution chosen will not completely eliminate the necessity for many temporary expedients and will take six to ten years to accomplish. Alternate waterfront facilities and their accompanying transit sheds and storage areas must be provided during this time in order that the millions of tons of commerce may continue to flow through the Harbor.

Economic Effect of Subsidence

The overall economic practicality of undertaking surface remedial measures is proven without question, although isolated cases present less attractive pictures economically. The reason for this overall finding is based upon the high value of the facilities involved and their importance to the entire West. Compared with the economic advantages to the public and the gross income of the area, subsidence remedial costs are reasonable. The overall corrective measures, while an economic burden, are nevertheless essential to the continued functioning of a number of very important services that are a part of the economic pattern of Southern California and the Pacific Southwest. The problems of these corrective measures are being met readily by private and public agencies alike, but must be well coordinated and require heavy financing.

Relationship of Subsidence to Tideland Grant

Probably all of the tidelands and submerged lands granted to the City of Long Beach by the State of California are within the subsiding area, but only the westerly portion presents a serious problem at this time.

The filled land area of the Long Beach Harbor in the westerly portion of the tidelands grant has been intensively developed for navigation, commerce and fishery, but this area is also near the center of the bowl of subsidence.

Piers A, B, C, D and E of the Long Beach Harbor and the U. S. Naval Station and Shipyard are within this area that has already subsided from 2 to 18 feet and is expected to reach a maximum of 30 feet before subsidence ends.

Continued operation of the highly valued installations of this area is necessary to the economic life of Long Beach and the surrounding areas.

Cost of Subsidence Expenditures

It is estimated that at least $33,200,000 has been spent by the Long Beach Harbor Department for subsidence up to June 1958. It is likely that the figure should be even larger, but until recently no attempt had been made to segregate subsidence costs in the City's accounts. As was true with past expenditures, the majority of prospective capital improvement projects includes very substantial subsidence costs amounting to an economic $30,000,000. Even this

figure does not include all future subsidence costs. It is impossible to consider improvements as separate from subsidence costs; by necessity any construction must remedy present subsidence and allow for future sinking.

Cost Figures Limited to Harbor Revenue Fund

The problems and costs referred to herein represent past charges to and future obligations of the Harbor Department of Long Beach in the Harbor District, as constituted in 1955.

Substantial subsidence expenditures outside of the Harbor District by other public agencies including the City of Long Beach are not included in the cost schedule.

Water Flooding Program

The Long Beach Harbor Department started a pilot water flood project in 1954 to study the use of filtered sea water for repressuring the subsurface oil formations. The complex structure of the Wilmington Oil Field has been studied by the Petroleum Division of the Harbor Department and by consultants employed by the Harbor Department and the City of Long Beach.

Water injection in the City-owned portion of the Wilmington Oil Field has been expanded to 119,200 barrels of water a day, and approximately 30,000,000 barrels had been injected to the end of 1958. The water flooding program is being expanded as rapidly as possible in order to prevent further drop in pressures in the oil zones.

The First Extraordinary Session of the 1958 State Legislature passed new subsidence control legislation that became effective July 24, 1958. This Subsidence Act determined that the people of the State of California have a direct and primary interest in arresting and ameliorating the subsidence and compaction of land in those areas overlying or immediately adjacent to producing oil or gas pools within the State where valuable buildings, harbor installations and other improvements are being injured or imperiled; that in certain of such areas of the State land already has subsided to a great extent and is continuing to subside at an alarming rate, resulting in injury to surface and underground improvements through land movement; that the results of studies by qualified engineers indicate that the only feasible method that can be expected to arrest or ameliorate subsidence in such areas is by repressuring subsurface oil and gas formations thereunder. The repressuring operations shall be under the jurisdiction of the State Oil and Gas Supervisor.

The Board of Harbor Commissioners established a Subsidence Control and Repressurization Division to coordinate the work of the Harbor Department, City of Long Beach and the other operators in the Wilmington Oil Field in the water injection program.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

LOS ANGELES COUNTY FLOOD CONTROL AND WATER CONSERVATION[a]

C. T. Newton,[1] M. ASCE and Harold E. Hedger,[2] M. ASCE

ABSTRACT

The extensive Los Angeles metropolis situated on a narrow and arid coastal belt receives occasional storms of high intensity. Problems of flood control planning, coordination, funding, design and construction have been complicated due both to the topography and heavy urban development. As a corollary to flood control projects, arrangements are effected to conserve a major portion of the limited runoff for replenishment of a dwindling ground water supply.

Flood Control

The Los Angeles area is remarkable! It is remarkable for its mild climate, for its topographical features, for its explosive population growth, and for its industries that are vital to the world of entertainment and the national defense. These very aspects, together with the foresight of enlightened public leaders in promoting measures to enhance the local economy, have resulted in a remarkable flood-control project to protect Los Angeles County. It is a long-range project, a vast undertaking and an outstanding example of close cooperation and participation between local, State and Federal agencies in a common cause. The two principal cooperating agencies in the program are the Los Angeles District of the U. S. Army Engineers and the Los Angeles County Flood Control District.

Note: Discussion open until November 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2069 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. WW 2, June, 1959.

a. Presented at the February 1959 ASCE Convention in Los Angeles, Calif.

1. District Engr., U. S. Army Engr. District, Los Angeles, Calif.
2. Retired Chf. Engr., Los Angeles County Flood Control District, Los Angeles, Calif.

Features of Area

Los Angeles County, in southern California, with Pacific Ocean frontage of 70 miles, is located some 120 miles northerly of the Mexican Border, and comprises about 4000 square miles. The San Gabriel Mountains, with peaks to 10,000 feet, are included just within the northern County boundary. The "Los Angeles County Drainage Area," (Fig. 1) about 1100 square miles in extent, comprises the watersheds draining to the Pacific and includes the coastal-plain area. Metropolitan Los Angeles occupies this coastal plain extending south from the mountain foothills to the ocean.

Although the region is arid, with an average precipitation of only 15 inches, major storms occur during the rainy season, November through April. Heavy rainstorms blow in from the Pacific Ocean and discharge large amounts of precipitation on the mountains and coastal plain. When this drenching occurs, a deluge of water and debris pours down from the mountain range and sweeps onto the 25 mile wide densely populated coastal strip.

The coastal plain has been subjected to severe flooding on many an occasion. There have been 26 floods of note since 1815. In fact it has been fairly well determined that the high water of 1825 was responsible for changing the main drainage channel to an outlet on San Pedro Bay from a previous mouth in Santa Monica Bay, to the North. This old watercourse very definitely had adverse affects on early Los Angeles, since it traversed the settled portion which was near the present "downtown area."

There are three principal drainage systems in the Los Angeles County Drainage Area. They are the Los Angeles River system, the San Gabriel River and Rio Hondo pattern, and Ballona Creek. Los Angeles River, the largest of the three and the main collecting channel for runoff from the high northerly, northwesterly, and central portions of the county, with a drainage area of 818 square miles, now flows into the ocean near the Los Angeles-Long Beach Harbor. The San Gabriel River receives runoff from the easterly and northeasterly mountainous parts of the county and enters the ocean near Alamitos Bay. Part of the flood flows from San Gabriel River are diverted at Whittier Narrows Flood-Control Basin into the Rio Hondo which flows in a general southwesterly direction and then joins the Los Angeles River about 12 miles from the ocean. Ballona Creek drains the southern slopes and adjacent plains of the Santa Monica Hills.

Since the mid 30's the Los Angeles area has had an explosive expansion in population and industry. Greater Los Angeles had about 2-1/2 million people in 1935; now the figure is about 6 million. Automobile registrations have increased from nine-tenths million to 3 million. Freeway mileage in Los Angeles County in 1935 was insignificant; now it measures 225 miles. Urban development has overrun farms and orchards to the extreme county borders. Incorporated municipalities in the county numbered 44 in 1935; now there are 61. Industrial development since World War II has been responsible for capital plant expansion of about two-billion dollars. The Los Angeles area is now one of the greatest economic centers in the country.

Inception of the Program

Attention was first formally directed to the problem of flood control about 45 years ago. Prior to 1914 there had been no attempt to formulate a definite

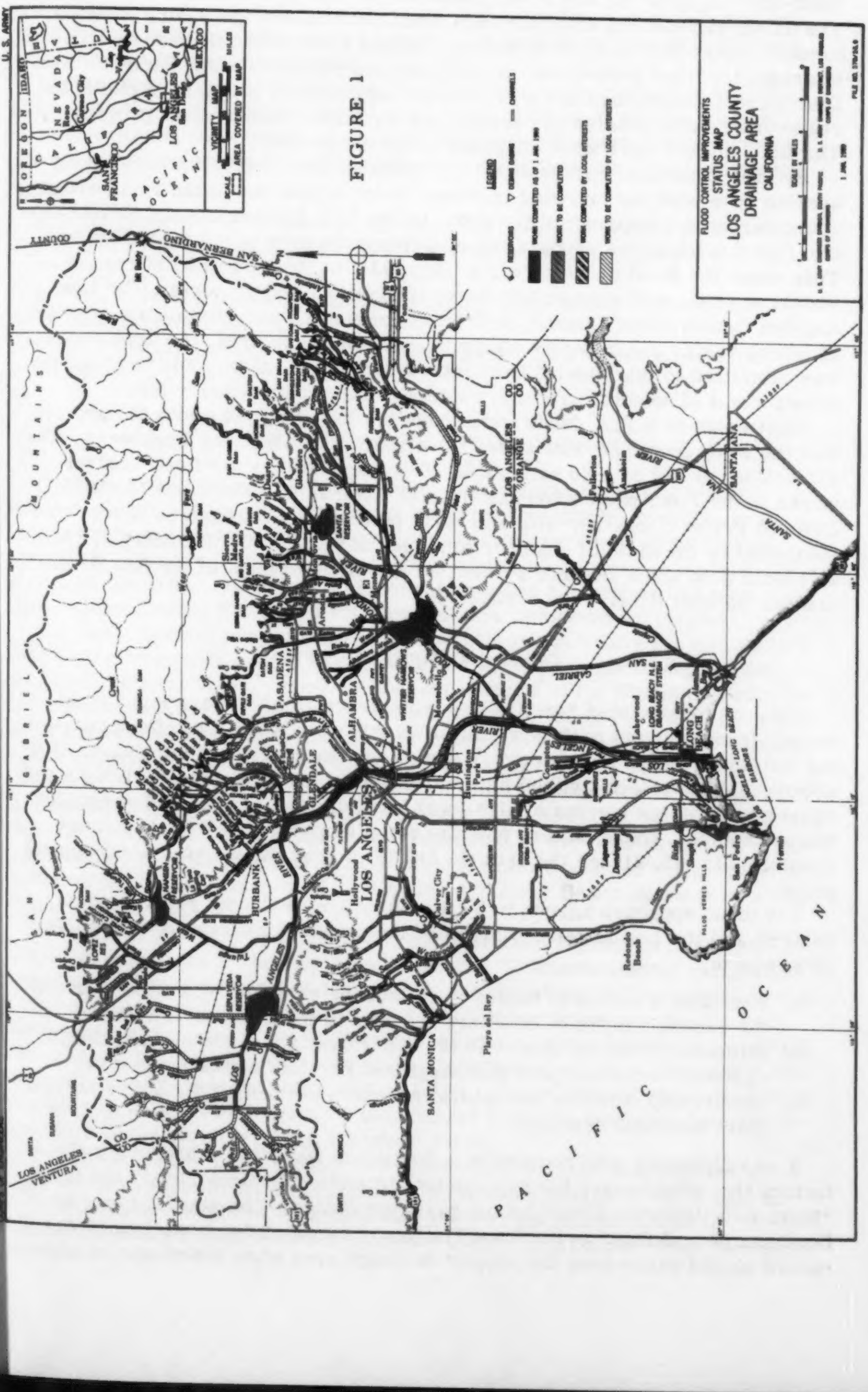

FLOOD CONTROL IMPROVEMENTS
STATUS MAP
LOS ANGELES COUNTY
DRAINAGE AREA
CALIFORNIA
LEGEND
VICINITY MAP
AREA COVERED BY MAP
U. S. ARMY
CORPS OF ENGINEERS
SAN GABRIEL MOUNTAINS
SAN BERNARDINO COUNTY
LOS ANGELES
VENTURA
LOS ANGELES
ORANGE
PASADENA
GLENDALE
BURBANK
ALHAMBRA
LOS ANGELES
Hollywood
Culver City
SANTA MONICA
Playa del Rey
Redondo Beach
San Pedro
LONG BEACH
Lakewood
Compton
Huntington Park
Montebello
Whittier
Fullerton
Anaheim
SANTA ANA
SANTA FE RESERVOIR
HANSEN RESERVOIR
SEPULVEDA RESERVOIR
WHITTIER NARROWS RESERVOIR
LOS ANGELES - LONG BEACH HARBORS
PACIFIC OCEAN

FIGURE 1

flood-control policy by any of the public or private agencies in the county. The damages resulting from the 1914 flood particularly the silting of newly dredged areas in Los Angeles Harbor, caused a concerted demand by local interests for flood protection. In 1915 the Los Angeles County Flood Control District was established and a permanent organization set up to perform the engineering work entailed in carrying out an active flood-control program. The flood of 1916 gave added impetus to the movement.

By 1930 it became apparent that the program of construction was barely keeping pace with the increase in storm-water runoff due to the rapid urban and suburban development of the area, so the Los Angeles County Flood Control District began the preparation of a comprehensive plan of flood control. Then came the flood of New Year's Day, 1934, in the confined LaCrescenta-Montrose area, with excessively heavy loss of life and property. The Los Angeles County Flood Control District commenced a program of construction of debris basins and concrete channels in the stricken area. An application was submitted to the Federal Government for W.P.A. allotments to speed the construction of selected protective items in the comprehensive plan.

Allotments of E.R.A. funds were made the following year with the proviso that the work should be done under the direction of the Army Engineers. The Flood Control Act of 1936 authorized 70-million dollars for flood-control works in the Los Angeles and San Gabriel Basins. On December 15, 1936, a Definite Project, together with a general plan of the Los Angeles River, was submitted by the district engineer to higher authority. On February 4, 1938, a general plan and a Definite Project for the improvement of the Rio Hondo and San Gabriel River were submitted.

General Design Criteria

Hazards to populated Los Angeles from heavy rainfall were found to be mainly from mud and rock flows in the foothills and swift flash floods exceeding the capacities of the waterways and spreading over the flood plains. Disaster could occur quickly; and high waters could subside rapidly. Long steady rains are rare, but storms of high local intensity can be expected during the winter months. These factors plus the steep slopes and relatively narrow coastal plain established the criteria for developing the general flood-control plans.

The basic approach agreed to by the Los Angeles County Flood Control District and the Los Angeles District of the Corps of Engineers, in the late 30's, was to:

a. Establish a series of basins to collect the mud and rock flow debris at the canyon mouths of tributary streams in the mountain foothills.
b. Build flood-control basins in the upper reaches of the main drainage systems to contain peak discharge and regulate downstream flow.
c. Rectify and stabilize the natural channels throughout the entire coastal plain for rapid drainage.

It was obviously also necessary to formulate the critical hydrological factors that would cause the hypothetical "Standard Project Flood" and the "Maximum Probable Flood" as the basis for design. The standard project flood has been defined as the flood that would result if the maximum storm of record should occur over the project drainage area when hydrologic conditions

were seasonably favorable for flood runoff. The maximum probable flood has been defined as the flood that would result if the maximum possible precipitation for the drainage area were to occur at a time when ground conditions are conducive to maximum runoff.

Design storms and flood criteria have been gradually refined in coordination with the U. S. Weather Bureau and from records of severe storms. Present criteria, by methods of transposition and with considerations as to type of storm and topography, take into account critical record storms such as:

a. The great storm of March 1938 when four days of intense precipitation fell on the Los Angeles area, already subjected to greater than annual average rainfall.
b. The severe 3-day storm of late January 1943, centered in the San Gabriel Mountains, and
c. The 3-hour thunderstorm of 3 March 1943 which broke all records for intensity of short-period rainfall.

Salient Aspects of Development

The Los Angeles County Flood Control District and the Los Angeles District, Corps of Engineers, by close cooperative planning over the past twenty-three years, have worked out coordinated long-range construction schedules. The program has been gradually expanded to keep pace with the growth of the area. Basic premises of the original comprehensive plan of the Los Angeles County Flood Control District have remained unchanged.

The flood-control program in the Los Angeles area really got rolling in 1935 with the impetus of the '34 disaster and the allotments of depression prompted Federal funds. The Los Angeles District of the Corps was rapidly expanded to assume the new flood-control function. Since then annual efforts of the County and the Corps have barely kept pace in providing flood-control works to protect the rapid growth described earlier.

Prior to 1935 the three basic stream patterns draining the area essentially followed their meandering natural water courses. However, between 1919 and 1921 the lower Los Angeles River was realigned by the construction of a four-mile channel in a southerly direction to the Pacific Ocean for the purpose of diverting flood flows from the Los Angeles and Long Beach Harbors at a cost of 1.1-million dollars.

With the first available funds under the Federal Program primary attention was directed to the Los Angeles River. Transcontinental railroads occupied the banks adjacent to the river in the downtown and industrial areas. Large and expensive industrial installations needing track service abutted the existing railroads. These factories, because of their location, prevented any railroad-track relocation. To make the situation more apprehensive the river also bifurcated the city of Los Angeles. These critical features forced the decision to start by improving the channel from the southerly city limits upstream to the vicinity of San Fernando valley.

Ballona Creek, which followed one of the many former Los Angeles River channels also came in for improvement in the early stages of the program.

For the next few years emphasis was placed upon getting Hansen, Sepulveda, Santa Fe and Whittier Narrows Dams constructed. This permitted

an orderly procedure of improvements of both the main channels and their tributaries upstream toward the flood-control dams.

After the Korean incident, efforts were concentrated on the completion of the Los Angeles River channel improvement. By the end of the 1957 construction program the objective was obtained and the main channel was stabilized from Canoga Park in the upper-western end of San Fernando Valley (Fig. 2) to the Pacific Ocean in Long Beach.

Most of the current construction work is being done on the tributaries of San Gabriel River and Rio Hondo systems, which lie in the San Gabriel valley to the east of Los Angeles and is considered to be a high priority area. However, this does not mean our planning and construction have been ameliorated; in this area suburban development still races ahead of flood-control improvements.

When the Los Angeles County Flood Control project is completed it will include the following items:

a. Los Angeles River Basin:

(1) 17 debris basins on 53 miles of tributary streams.
(2) 3 major flood-control basins (Sepulveda, Hansen, and Lopez).
(3) 48 miles of main channel control.
(4) More than 100 bridges over main and tributary channels.

LOS ANGELES RIVER, SAN FERNANDO VALLEY

FIGURE 2

b. San Gabriel River and Rio Hondo Basins:

(1) 14 debris basins.
(2) 2 major flood-control basins (Santa Fe and Whittier Narrows).
(3) 45 miles of main channel improvement.
(4) 104 miles of tributary channel work.
(5) More than 200 bridges over main and tributary channels.

c. Ballona Creek Basin:

(1) 2 debris basins.
(2) 23 miles of main and tributary channels.
(3) 15 bridges over main channel.
(4) 2 jetties at the mouth.

Selected Design and Construction Features

The steep slope of the coastal plain produces high-velocity runoff from both the mountain and valley areas. To protect the intensely developed industrial and residential areas, concrete-lined channels are required to confine and control high-velocity flows. Near the base of the mountains, channels constructed on slopes of 4, 5, and 6 per cent are not uncommon. Velocities in such channels become as high as 50 feet per second.

A great amount of hydraulic-laboratory investigation has been done to develop design criteria to cope with the problems inherent in supercritical flow around curves, through bridges, (Fig. 3) and at confluences. Rights-of-way limitations often make it necessary to introduce considerable curvature in channel alinement; short-radius curves require superelevation of the channel bottom in order to maintain equilibrium in the flow. For example, from experience gained during the 1938 flood it appeared that curve radii in a major channel should not be less than 2,000 feet. After knowledge was gained from hydraulic-model studies of the problems, a 3,600-foot reach of the upper Los Angeles River was designed and built as a rectangular reinforced concrete channel with only 276 feet on tangent and a minimum radius of curvature of 382 feet.

In the design of channels, consideration is given to the use of both rectangular and trapezoidal cross section. Trapezoidal channels usually are less costly to construct, but require more rights-of-way, because of greater width and greater radius of curvature need; also bridge costs are higher due to longer overall lengths. Sometimes a transition from a trapezoidal cross section to a rectangular cross section is found desirable in order to utilize a certain existing bridge. Even the addition of upstream pier extensions to the existing piers or a special design for the configuration of the proposed channel section will preserve the bridge.

To develop a large network of flood channels for this large metropolitan area it is necessary to continually join smaller to larger channels. Merging two high-velocity streams flowing 30 to 40 feet per second can be quite complicated with consequent acute design problems.

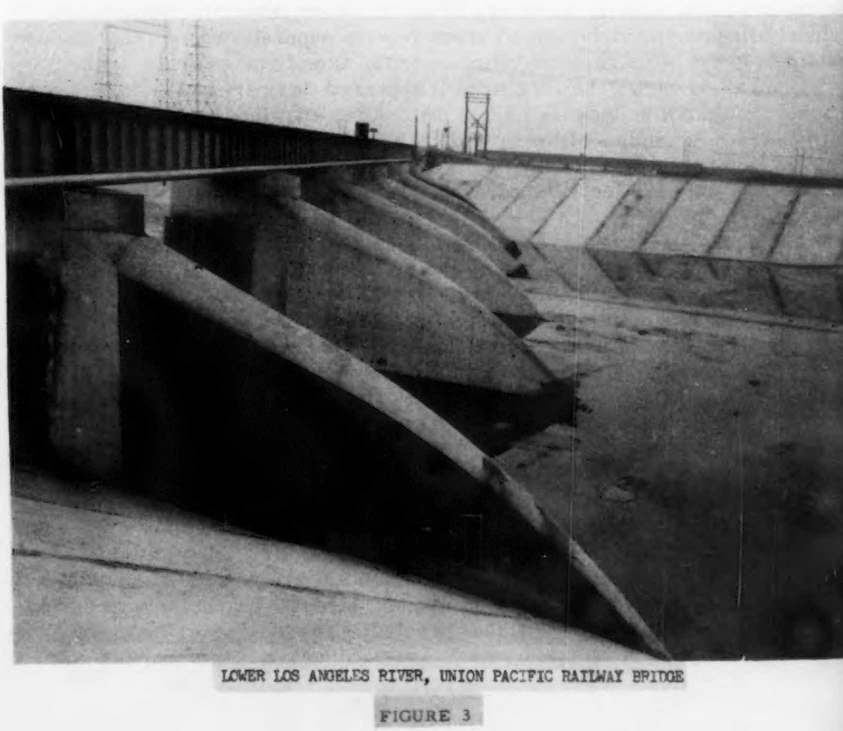
For some of the smaller channels in the foothill areas, with steep slopes, narrow rights-of-way, and crooked alinement, circular channel sections of cast-in-place concrete are used, thus precluding the need of a tilted invert for an extremely short radius.

The retaining walls of the open rectangular concrete channel sections are designed as L-type walls and constructed in pairs opposite each other with the wall bases forming the channel invert or abutting against the invert slab, depending on the channel width. The walls are designed for two main limiting conditions, with the channel empty and with the channel full.

Various types of channel subdrainage systems have been employed for several years. The current method of collecting ground water is briefly described as a system comprising a pervious drainage blanket, perforated collector pipes, manholes, and flap gates.

The uptown reach of the river was improved during 1938-39 as a trapezoidal section, with grouted-stone side-slope protection, and a heavy concrete thrust block at the toe of the slope. Because much of this reach had been set aside for the city of Los Angeles by a court decision for water percolation and reclamation the invert was left unpaved with transverse stabilizers spaced at about 3,000-foot intervals. The velocity of the stream in this area reaches 18 to 20 feet per second. Observations, made over an extended period, indicated that means need be developed to secure the levee slope protection, which was becoming vulnerable to streambed erosion. The scheme adopted was to pave the invert with a layer of cobblestone properly graded, approximately 30 inches in thickness, of a size capable to resist displacement by stream flows yet permit percolation to the substrata.

Several time-saving construction features, in line with technological progress, have been adopted. Prestressed or precast concrete highway and



LOWER LOS ANGELES RIVER, UNION PACIFIC RAILWAY BRIDGE

FIGURE 3

railroad bridge members are frequently used. To minimize interference with rail traffic, where a covered section of channel improvement is being constructed, jacking of a substantial length of a precast section is resorted to. Concrete channel invert paving is now placed with continuous reinforcing steel.

Economic Aspects

When the first comprehensive analyses of costs and benefits for the overall project were made in the 30's a ratio of benefit to cost of 1.52 was determined. With an expanded plan, growth of the area, increased property values and with due consideration for increased costs, the present estimated benefit-cost ratio for the overall project is 3.60.

The comprehensive plan is now considered to be somewhat less than two-thirds complete. The current estimate of Federal cost is 370-million dollars with about 170-million dollars of work remaining to be done. Even though the remaining items to be improved include several debris basins, miles of small channels and rectification of the greater portion of the San Gabriel River, the completed flood-control works have already prevented estimated flood damages of more than 151-million dollars. Annual congressional appropriations have recently been averaging between 16- and 18-million dollars. Some construction work for local agencies in conjunction with the flood-control improvements is habitually performed under Federal jurisdiction with contributed funds; amounting to approximately 2-million dollars per annum. The yearly construction season being confined to the dry summer months the several separate construction contracts are usually held to amounts of 2-million dollars or less to assure completion. With this rate of annual funding the Los Angeles County Drainage Area project should be completed in about another 10 years.

This, in brief, is the story of the U. S. Army Corps of Engineers' part in helping to solve flood-control problems in Los Angeles County.

The Corps of Engineers, naturally, takes pride in the many dams, and miles of channel that have already been designed and constructed. But, beyond that, the Corps sincerely appreciates the opportunity afforded by Congress to participate with local interests in a combined effort for enhancing the security and economic well-being of a tremendously vital and nationally important center of population and industry, Los Angeles.

Water Conservation

The Los Angeles County Drainage Area Project heretofore described comprises the major river channels and larger tributaries of the metropolitan part of Los Angeles County. As a Federal project it is handled as a joint responsibility by the Los Angeles District of the Corps of Engineers and the Los Angeles County Flood Control District. The former conducts the planning, design and construction of the component parts of the project in accordance with Congressional appropriations. The Flood Control District joins in the planning, procuring of rights of way, financing the reconstruction and relocation of bridges and highways (which at times approaches the cost of the channel construction), and assumes responsibility for maintenance and operation of the channel improvements subsequent to their completion by the Corps

of Engineers. Funds expended by the District for the right of way and highway reconstruction costs are reimbursed by the State of California.

This project, which is a major undertaking requiring approximately thirty years or more to accomplish, will provide a system of flood channels which will constitute the backbone of the watershed drainage complex which contributes flood runoff to it. Excluded from the project are a considerable number of natural watercourses which did not meet the Federal criteria as to cost-benefit ratio when the project was approved by Congress in 1941 but have subsequently become a considerable source of flood hazard due to rapid development of adjacent properties in the intervening years. The improvement of these channels is therefore, the sole responsibility of the Flood Control District.

The development of a system of supplemental channels or storm drains which will collect storm waters close to their source and convey them to the main system without damage to adjacent property is a local responsibility. Actually, the main system cannot be utilized to its full capacity until such an appurtenant system is completed. It may be surprising to note that the estimated cost of such supplemental drainage will eventually exceed one billion dollars, a much larger figure than the overall financial requirements for the main flood control network. To the present time, the Flood Control District has financed, through authorized bond issues, over four hundred million dollars for the construction of these storm drains.

The conservation of its flood waters is another primary responsibility of the Flood Control District. This problem, which deals with the replenishment and safeguarding of the ground water resources of the District, is often accentuated by improvement of the flood channels described heretofore since many of these channels are concrete lined, preventing or restricting the natural percolation of flood waters which had been experienced in the past.

Hence, there is a large field of flood control and conservation activity in Los Angeles County in addition to the basic flood control system which will be provided by completion of the Los Angeles County Drainage Area Project, and this has necessitated creation of a major local agency to work with the Corps of Engineers and, at the same time, solve the problems which lie outside the Federal jurisdiction.

There is little difference in the methods used by the Corps of Engineers and the Flood Control District in the development of flood control and drainage projects and, therefore, the remainder of this paper will be devoted primarily to the conservation activities of the Flood Control District which are conducted as a partner of flood control.

Passage of an Enabling Act by the California State Legislature in 1915 created the Los Angeles County Flood Control District. In addition to its flood control responsibilities, the District was directed to "conserve such waters for beneficial and useful purposes by spreading, storing, retaining or causing to percolate into the soil within said District".

Administratively, the District has recognized its dual mission by creating a Flood Protection Branch and a Dams and Conservation Branch with both reporting directly to the Chief Engineer.

An examination of topographic and climatic conditions in Southern California reveals the close relationship existing between the two functions and the vital necessity for each. An intensely populated region such as Los Angeles County with a relatively narrow flood plain lying at the base of steep mountain slopes obviously requires a well developed and highly efficient flood

control system. Additionally, the semi-arid nature of the basin area, with rainfall concentrated in the fall and winter months, and with a water importation program unable to fulfill its ultimate requirements, indicates the high priority which must be assigned to the conservation of water from every available source.

Research has revealed that the most efficient method of water conservation in the County has been through the recharge of ground water basins, utilizing percolation in natural stream channels and in artificial spreading grounds and basins.

In meeting its twin responsibilities of flood control and water conservation, the District in the years following its creation began the construction of 14 major dams. These reservoirs are located in or below principal canyons, controlling a mountain watershed of approximately 400 square miles. Final units of this program were finished in 1939 with the completion of construction on the San Gabriel Project. This undertaking included both the San Gabriel and Cogswell Dams and offers substantial protection to the San Gabriel watershed area. Subsequently, six major reservoirs have been constructed by the United States Army Corps of Engineers and supplement District dams as a defense against major flood inundation in the Coastal Plain.

It should be noted that District dams are operated in full accordance with legislative requirements; i.e., both for flood protection and the conservation of flood waters. Under provisions of the Federal Flood Control Act of 1936, Corps of Engineers facilities are directed primarily at the control of flood waters. However, the reduction of flood peaks through temporary storage in the installations constructed by the Federal agency has a beneficial and positive conservation result. The release of controlled flows, relatively free from debris, makes it feasible to divert a portion of those flows from stream channels to spreading grounds and basins as a replenishment measure.

Earlier in the paper, the paving methods utilized by the Corps of Engineers as a part of the channel improvement program in this County were described. The District concurs with this approach but has systematically examined the percolation loss attendant to such paving. In the interests of sound conservation practice, we feel that this loss must be a subject of frequent study so that appropriate corrective conservation measures may be added to the design of further channel improvements.

Despite a competently engineered importation program, Los Angeles County remains dependent upon ground water basins for approximately 55% of its supply. There are seven distinct major ground water basins in the County which include more than 28 sub-basins with a total surface area of over one million acres. In excess of 65% of these basins are currently overdrawn as a direct result of an extended period of subnormal seasonal rainfall and the unprecedented expansion of industry and population in the area. It is obvious that a positive approach to this problem must include artificial replenishment of these depleted basins by utilizing both storm waters and surplus imported waters. Development of an effective importation system has in no way diminished the extreme value of these underground reservoirs particularly when the possible effects of an interruption of the aqueduct flows are considered. A prolonged drought condition in Southern California during the past fourteen years, with only two seasons as an exception, has materially reduced the amount of natural replenishment of the ground water basins. However, even though this deficiency of rainfall had not occurred, artificial replenishment would have been necessary owing to the present and sustained heavy draft on underground water resources.

Flood Control District dams are so operated that they function in the achievement of three primary purposes. First, they are utilized for the extraction of debris, both lighter and heavier than water, from recurring storm precipitation. Secondly, they regulate storm waters through temporary reservoir storage and provide for their release at a reduced rate in relation to the inflow peak. Finally, the dams, whenever safety factors permit, conserve the flood waters to an extent whereby clarified waters can be released at an appropriate rate to permit percolation in the stream channels and spreading grounds.

To date, the District has constructed fourteen off-channel spreading grounds and basins (Fig. 4) containing a combined gross acreage of approximately 1400 acres and with a combined percolation capacity of about 1370 cubic feet per second. In addition, it cooperates with other agencies in the operation and maintenance of six other spreading grounds with a total area of nearly 1400 acres and a combined percolation capacity of approximately 475 cubic feet per second. Future plans call for the development of an additional 425 acres in conjunction with channel improvement and storm drain construction. Essentially, these spreading grounds consist of canals and shallow basins composed of highly porous sands and gravels, usually adjacent to river channels, into which water can be diverted for seepage into the underground water reservoirs.

In supplementing its ground water replenishment program, the Flood Control District has also utilized abandoned gravel pits and, wherever feasible, is continuing to acquire these pits to augment spreading operations.

SPREADING GROUNDS NEAR PASADENA

FIGURE 4

It has been estimated that if regulated flood waters were available to supply these basins and pits to capacity, they would be able, initially, to absorb a flow of approximately 1800 cubic feet per second in the aggregate. The continuing capacity of the grounds depends upon the pumping extractions from the effected ground water basin. In most cases these extractions, currently, are quite sizable.

Importance of water conservation works in relation to stream flow in our upstream ground water basins is emphasized by the small waste to the ocean attributable to these sources. Of an average of about 750,000 acre feet of rainfall in the area upstream from Whittier Narrows, only 38,000 acre feet per year, or 5%, wastes to the ocean. On the other hand, in the lower areas where there is little or no opportunity for storage or retarding high flows and where the ground is generally impermeable, it is impossible to conserve an appreciable amount of the storm deposits.

Studies by hydrologists have indicated that as a long time annual average about 20,000 feet of flood waters which otherwise might be lost to the ocean will be conserved each year in the stream channels and spreading grounds which have been developed in the Central Basin. This would constitute a normal supply for approximately 100,000 people in this area alone.

Recognition was accorded the increasing need for using imported waters as an additional spreading source with an amendment to the Los Angeles County Flood Control Act in 1951 which authorized the District's Board of Supervisors to establish water conservation zones. The amendment provided that an ad valorem tax could be levied within the affected areas for the purchase of reclaimed or imported water as a conservation measure. Subsequent to the amendment to the Flood Control Act, two such conservation zones have been established by the Board of Supervisors. Zone I was created in 1952 and, generally, includes the Central Basin of Los Angeles County. Special attention was given to the West Coast Basin of the County with the establishment of Zone II in 1954.

Since the creation of Zone I and up to February 1, 1959, the District has purchased in excess of 300,000 acre feet of untreated Colorado River water for percolation into the ground water basins. Geographically, Zone I roughly embraces an area bounded by the Montebello Hills on the north, Long Beach on the south, the Los Angeles-Orange County boundary line on the east and the Los Angeles River on the west. Water purchased from the Metropolitan Water District and released from its La Verne-Garvey feeder into the San Gabriel River and Alhambra Wash, a tributary of the Rio Hondo, flows through the Whittier Narrows Reservoir of the Corps of Engineers and seeps into the porous soil of the stream channels and spreading grounds of the Montebello Forebay. The Rio Hondo spreading grounds, comprising 429 acres on the east side and 130 acres on the west side of the Rio Hondo Channel, are the largest owned and operated by the District. Supplementing these spreading grounds are 118 acres on the west bank of the San Gabriel River and approximately 150 acres of permeable stream channel adding up to a total recharge area of some 827 acres. This acreage is capable of absorbing approximately 600 cubic feet per second of continuous flow.

As a result of the replenishment practices initiated along the San Gabriel River and the Rio Hondo, in excess of 1,500,000 acre feet of imported and natural water has been conserved.

Zone II, the West Coast Basin, includes an area along the coast of Southern California from Playa del Rey to the Palos Verdes Hills and extends inland

for approximately seven miles. Funds levied in this zone are used for the purchase of imported water which is utilized by the exploratory wells and appurtenances of the West Coast Basin Barrier Project (Fig. 5). Cooperating in the establishment of this project in 1952 and 1953 were the State of California Division of Water Resources and the Water Resources Board. It was the essential purpose of the program to determine the feasibility of injecting purchased fresh water into recharge wells to form a pressure ridge and thus bar further saline intrusion. Injection of this fresh water into the aquifers of the ground water basin, known as the Silverado Zone, has to date produced very gratifying results.

The problem of sea water intrusion and its resultant contamination of fresh ground waters has been the subject of study during the past 30 years in California. A survey by the California Division of Water Resources in 1950 revealed that 20 out of a total of 35 important coastal ground water basins were being subjected to serious saline intrusion or were in imminent danger of such contamination. Fifteen other coastal basins were described as potentially in danger of degradation. This report listed the West Coast Basin as the most critically affected in the entire state and this, quite naturally, was influential in initiating the experimental work in this area.

The Flood Control District had conducted several investigations of the water wells pumping along the west coastal area during the 1930's and developed evidence of the threatened saline intrusion which was then commencing.

The West Coast Basin has been defined as a pressure basin in that the water bearing zone is covered by a clay cap of considerable thickness. This

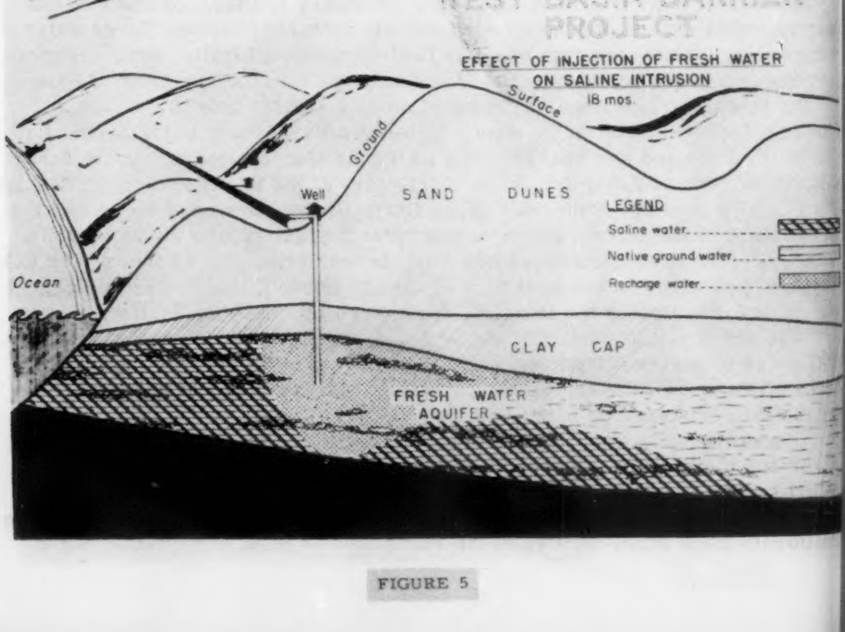


FIGURE 5

heavy and impenetrable clay topping prevents water spreading replenishment measures inasmuch as the water would be intercepted before reaching the ground water basin. Hence, the use of wells which are capable of piercing the clay covering and permit the injection of fresh water into the underground reservoirs has been necessary.

Salt water intrusion at Manhattan Beach had reached a point where it became necessary for that community to abandon eight of its producing wells. Estimates indicated that the intrusion in the area took place at the rate of about 500 feet a year. At the time the injection project was undertaken by the Flood Control District, sea water had intruded approximately one mile from the ocean.

In developing the District's Zone II experimental program, recharge wells were situated parallel to the ocean about 2,000 feet inland. Ground water at this point was six to twelve feet below sea level and was virtually pure sea water. It averaged 16,000 parts per million chlorides as compared to 18,250 parts per million for sea water. Nine recharge wells were put down with a space interval of approximately 500 feet and a total of 36 observation wells were also drilled. As operations progressed, additional recharge and observation wells were installed. This resulted in the combined observations of some 60 wells in the area.

Efforts to create a fresh water pressure barrier as a deterrent to further saline intrusion were begun in February 1953. Evaluation in November of that year revealed that a continuous fresh water line above sea level was being maintained along the 4500 foot test reach. Studies have indicated that of the fresh water injected into the subsurface area, little or none may eventually be wasted to the ocean. For the time being practically all of the injected water flows landward as a replenishment of the ground water basin and that oceanward movement has only traversed 1000 feet of the 2000 feet in five years.

Currently, the Flood Control District is operating this established recharge line on a standard maintenance basis. As a result of these operations, a strong barrier to further saline intrusion has been created along approximately one and one-half mile of the ocean front. Since initiation of the project, the injected fresh water has been detected 5000 to 6000 feet inland.

The District's recharge program along the West Coast Basin has suggested several important conclusions. It has been determined, for example, that in areas of comparable geological and hydrological conditions saline contamination can be prevented and controlled by recharge of the basins through injection wells. The pre-existing landward gradient between the ocean and the recharge wells may be reversed and the prevention of further sea water contamination accomplished through the recharge procedure which pressurizes a confined aquifer continually. In addition, the utilization of the recharge method will provide significant replenishment to inland ground water basins with only a minor loss of fresh water to the ocean, if any.

The project has been entirely successful and it would appear inevitable that similar measures will have to be undertaken over the eleven mile reach from the Ballona Escarpment to the Palos Verdes Hills. Similarly, studies are currently in progress concerning the efficacy of a parallel program of this type in the Dominguez and Alamitos gap areas. It has been estimated that approximately twenty-five miles of the coastline in Los Angeles County are threatened by saline contamination and it would seem apparent that recharge of affected fresh water basins will ultimately be required.

District conservation experiments have also included the employment of seepage pits in those areas where the presence of hard pan or other impervious materials has rendered the use of spreading from shallow basins ineffective. In essence, they are composed of bucket drilled pits at least 30 inches in diameter extending through the impervious layer into the permeable material. Thoroughly washed pea gravel of uniform size is placed in these pits which is then topped with a mound of gravel and coarse sand extending above the ground surface. This facilitates the periodic removal of silt and other materials, eliminating the necessity of disturbing the gravel in the pit proper. A test installation was developed by the District near the City of El Segundo. The pits were drilled about 40 feet apart with a total of 25 such pits within an area of one acre.

Engineers studying the problem of water replenishment in Los Angeles County have estimated that to maintain an effective fresh water barrier over the entire reach of sea coast from the Ballona Escarpment on the north and the Palos Verdes Hills on the south, a distance of approximately eleven miles, would require a continuous flow of between 75 and 100 c.f.s. to the injection wells. Review of the situation indicates that such an amount of imported water may not be continuously available. In addition, the cost of imported water is an important factor. Recognizing this condition, the District, in cooperation with the Board of Public Works of the City of Los Angeles, has been experimenting with the feasibility of using the reclaimed waste waters from the Hyperion Sewage Treatment Plant as a source for additional spreading and injection. These experiments have been promising and may supply a supplemental water reserve for the conservation practices of the District and other concerned agencies.

Reclamation of waste waters was the subject of a report filed with the Los Angeles County Board of Supervisors in November 1958. This report was prepared jointly by the County Sanitation Districts and the Flood Control District. It was the culmination of a study by the agencies of the potential reclamation of sewage now wasting to the ocean in Los Angeles County. Research indicates, according to the report, that "of the 465 million gallons per day of sewage presently discharging to the ocean, 280 million gallons are of such quality and available at such locations as to be amenable to reclamation and reuse." Economically, the report points out that the water is valued at $20 an acre foot which would indicate that the 280 million gallons reclaimed daily is worth $16,800. The joint report urged that the Board of Supervisors endorse the reclamation and reuse of waste water, that it sponsor legislation permitting the County of Los Angeles to finance the construction of water reclamation plants and that the waste water reclamation program envision the building and operation of a demonstration plant at the Whittier Narrows Reservoir area to provide ten million gallons daily of reclaimed water for ground water recharge.

The remarkable growth of population and industry in Southern California in recent years has accelerated the need for an early solution to the flood control and water problems characteristic of the area. Steady progress has been made by the Corps of Engineers and the District in the construction of flood control works capable of protecting the heavily settled Los Angeles Basin, and our agency, since 1952, has been engaged in a storm drain program designed to correct many interior drainage conditions which have been intensified by this pattern of phenomenal expansion.

As new solutions are proposed to the water problem in the area, the conservation practices developed by the Flood Control District are of increasing importance. For example, the recommendations by the State Department of Water Resources concerning the Feather River importation program includes provisions for the utilization of the depleted underground basins as equalizing reservoirs to meet seasonal or peak demands. Their importance is emphasized further when the effects are examined of any interruption of the water flowing into the area through the aqueduct systems as the result of war or natural disaster.

The Flood Control District considers the practices enumerated in this paper dealing with ground water replenishment, the protection of subsurface supplies and the development of additional water sources vital to the continued growth, well-being and safety of Los Angeles County.

P

An
sugge
fundar
paper
applic
bathry

Lit
proble
are fu
cerni
statis
ing th
A mi
gravi
uniqu
requi
effect
be us
relat
the b
zone,
stabi

Note:
a
pa
of
1. C
D

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

LITTORAL DRIFT IN VICINITY OF CHARLESTON HARBOR

James Neiheisel[1]

SYNOPSIS

An integrated method employing wave refraction and sediment analysis is suggested as an approach to the interpretation of littoral drift characteristics fundamental to coastal engineering problems. Correlations introduced in this paper are applicable to the central South Carolina coast but techniques are applicable to any coast where diagnostic minerals occur and reliable wind and bathrymetric data are available.

INTRODUCTION

Littoral drift is one of the many factors considered in coastal engineering problems. General knowledge of littoral currents and sediments in an area are fundamental to determination of direction of littoral drift. Much data concerning the nature of the forces operating in an area may be gained from wind statistics and by analysis of wave refraction diagrams. Information concerning the sediments may be derived from the presence of a diagnostic mineral. A mineral may be considered diagnostic if its relative abundance, specific gravity, particle shape, particle size, or response to applied forces are unique. Along the central South Carolina Coast, hornblende fits these prerequisites and in conjunction with wave refraction studies may prove to be an effective tool in understanding sediment transport. Before any mineral may be used effectively as an indicator, however, it is necessary to establish its relationship to its environment and to the other co-existing minerals. Perhaps the best proving ground is the coastal environment, particularly the littoral zone, where mineral concentration is related to sediment supply, coastal stability, and the forces of wind and waves.

Note: Discussion open until November 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2070 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. WW 2, June, 1959.

1. Chf., Petrographic Section, Concrete and Bituminous Branch, South Atlantic Div. Testing Lab., U. S. Corps of Army Engrs., Marietta, Ga.

The suggested method could serve as a foundation for more advanced techniques of measurement by coastal engineers.

General Coastal Considerations

A near static sea level exists along the South Carolina coast. R. J. Russell believes the main rise of sea level took place between 18,000 and 5,000 years ago as based on carbon isotope studies of wood samples from Mississippi alluvium. Since then the fluctuations of the sea have been minor with a maximum of 2.5 inches recorded mainly between 1930 and 1950.(1) With this concept of a near static sea level, it is possible to view sediment movement along a coastline more objectively in relation to forces of wind and waves.

The orientation of the South Carolina coastline is generally N.E. - S.W.; however, deviations occur in the vicinity of inlets, along areas of strong accretion, and along headlands of strong erosion. As will be demonstrated later, orientation of coastline along with accurate bathymetric data is especially important when considered in connection with wave refraction analysis used to estimate direction of littoral currents. Counter currents and unequal distribution of wave energy may be indicated in areas of strong erosion or accretion where marked deviation of coastline occurs.

Meteorological and Hydrodynamic Factors

Strong northeasterly winds prevail during the winter months and lighter southeasterly winds are dominant during the summer months along the South Carolina coast. A wind diagram of force, direction, and duration of winds operating in the Charleston, South Carolina area is shown in Fig. 1. From this statistical information it is observed that winds from the northeast constitute the strongest force and in terms of vector quantities represent the net wind force for seaward approaches to the coastline. Statistical wind data or "hindcasting" is fundamental to wave refraction diagram construction as set forth in U. S. Army Corps of Engineers, Beach Erosion Board, Technical Report No, 4. of 1954. Wind generated waves tend to become parallel with the shoreline as a result of "feeling bottom" in compliance with existing underwater topography. They break at a slight angle to the shore under most conditions with the result that a littoral current is induced. It is this current combined with the agitating action of the breaking waves that is the primary factor in causing sediment movement along a coastline.

An example of the usefulness of the wave refraction principle is shown in Fig. 2 for a portion of the coast south of Charleston, South Carolina. Wind generated waves from the southeast and east with a wave period of 5 seconds are used to approximate average conditions. A wave orthogonal interval of one mile is used from deep water and the orthogonals are graphically refracted to the beach in accordance with existing bottom topography. It can be readily observed that waves from the east produce southwesterly littoral currents and that waves from the southeast produce essentially no current (Fig. 2). Along this section of coastline it then becomes apparent that wind generated waves which approach from the east or northeast generally produce southwesterly littoral currents and those waves which approach from the south or southwest result in northeasterly littoral currents. Further, where

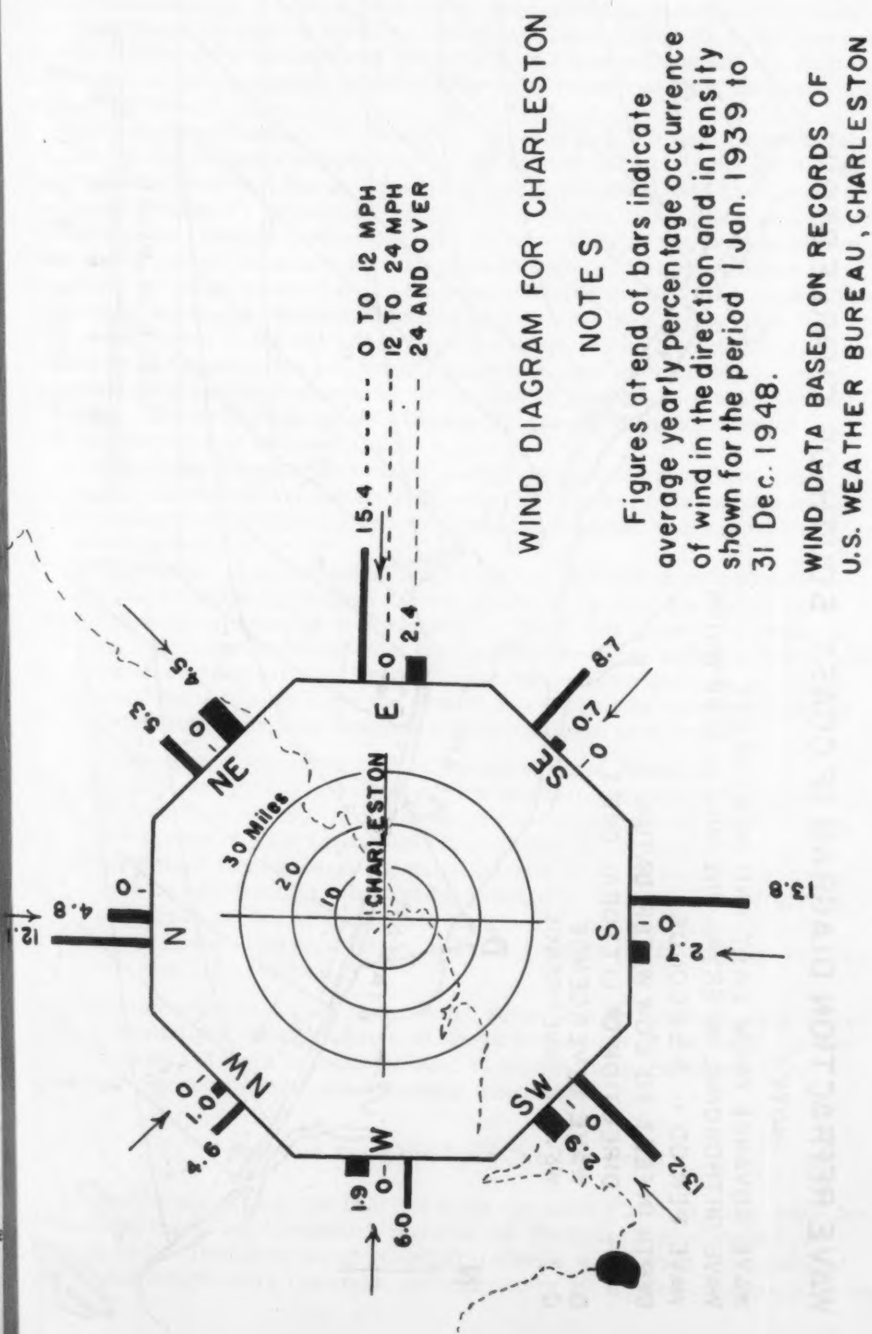

WIND DIAGRAM FOR CHARLESTON
0 TO 12 MPH
12 TO 24 MPH
24 AND OVER
NOTES
Figures at end of bars indicate average yearly percentage occurrence of wind in the direction and intensity shown for the period 1 Jan. 1939 to 31 Dec. 1948.
WIND DATA BASED ON RECORDS OF U.S. WEATHER BUREAU, CHARLESTON
N
NE
E
SE
S
SW
W
NW
CHARLESTON
30 Miles
20
10
15.4
0
2.4
8.7
0.7
0
13.8
0
2.7
13.2
0
2.9
6.0
0
1.9
4.6
1.0
0
12.1
4.8
0
5.3
0
4.5

FIGURE 1

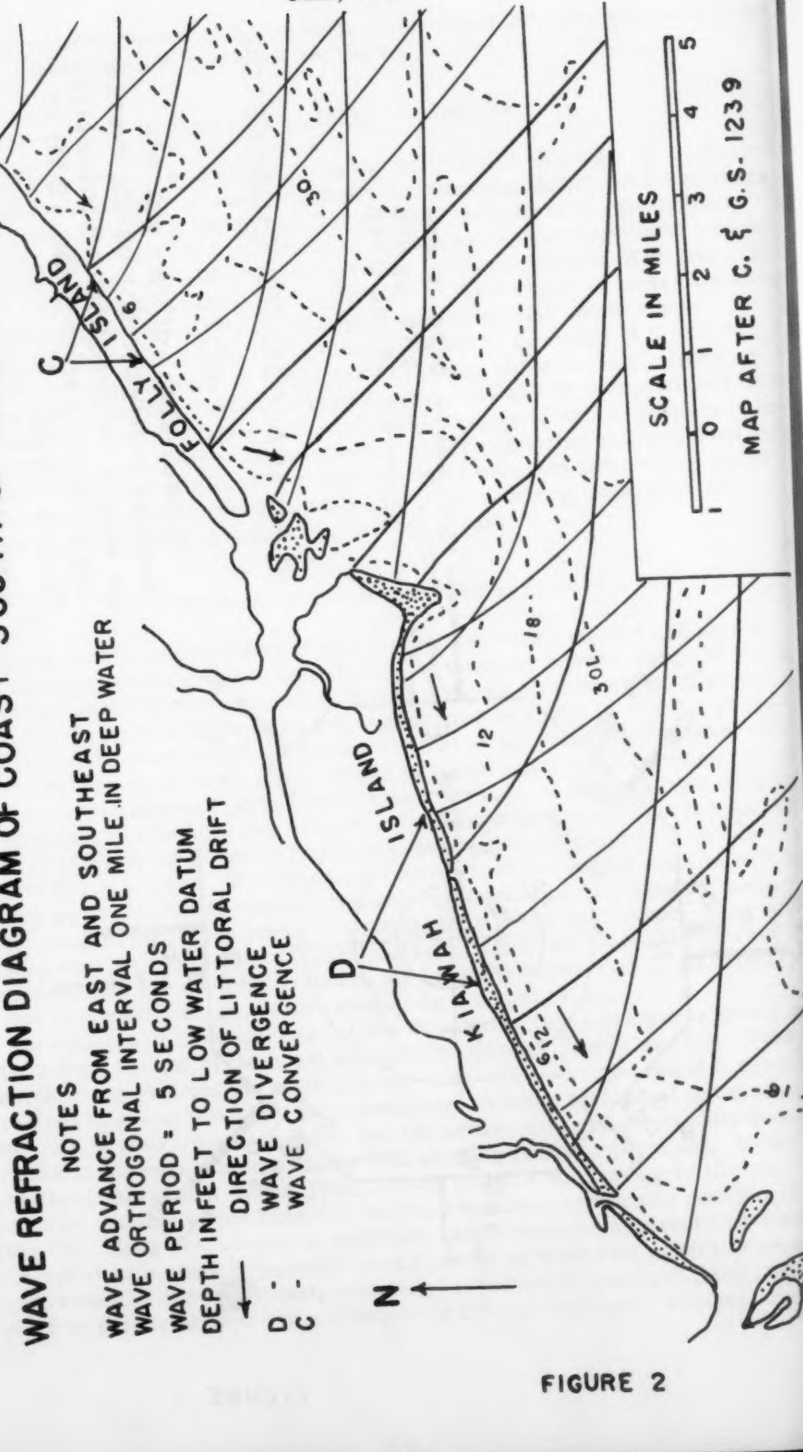

WAVE REFRACTION DIAGRAM OF COAST SOUTH OF CHARLESTON
NOTES
WAVE ADVANCE FROM EAST AND SOUTHEAST
WAVE ORTHOGONAL INTERVAL ONE MILE IN DEEP WATER
WAVE PERIOD = 5 SECONDS
DEPTH IN FEET TO LOW WATER DATUM
DIRECTION OF LITTORAL DRIFT
D - WAVE DIVERGENCE
C - WAVE CONVERGENCE
N
FOLLY ISLAND
KIAWAH ISLAND
C
D
SCALE IN MILES
0 1 2 3 4 5
MAP AFTER C. & G.S. 1239

FIGURE 2

wave orthogonals converge, higher waves are formed and where orthogonals diverge, lower waves tend to exist. From the diagram (Fig. 2), it is observed that the latter condition exists in the vicinity of Kiawah Island and the former is present along the central portion of the Folly Island coastline for waves approaching from the east.

In the vicinity of inlets, such as occur between Bull, Capers, DeWees, Isle of Palms, and Sullivans Islands, the coastline trend deviates from the normal and littoral currents differ in direction in compliance with this change. The sediment transport regimen also changes in response to the changes in coastal orientation and littoral current energy patterns. Some of the sediments from the moving littoral stream are drawn through the inlet and deposited in an inner bar or shoal and are effectively removed from the littoral stream. The remaining sediments continue downdrift by following the arcuate outer bar or as a consequence of the ebb and flood of the tidal cycles. The shape and dimensions of the outer bar are heavily dependent upon the littoral currents, the quantity of moving sediments, and the flow of flood and ebb tides through the inlet. Downdrift from an inlet the normal coastal trend and littoral current pattern are achieved again.

It is not clear whether the change in orientation of the shoreline causes the change in littoral current pattern or whether the littoral current pattern causes the change in orientation of the shoreline or if the two causes or effects can be separated.

The results of wave refraction diagrams may be used as an indicator to predict the direction of littoral currents produced by wind generated waves and the directional changes of wave advance which may occur as a result of differing bottom conditions or inlet tidal currents. These differing conditions for sections of coast are also related to processes of erosion and accretion.

Hydrodynamic factors along the central South Carolina coast as revealed by wave refraction diagram analysis are summarized as follows:

(1) Predominant littoral drift is in a southwesterly direction.
(2) Strong wave orthogonal convergence, indicating a greater magnitude of applied wave energy, is observed on eroding sections of coastline and divergence of wave orthogonals is observed in areas of accretion.
(3) Deviations from general coastal alignment occur in areas of strong erosion or accretion. Headlands, exposed to strong northeasterly wave attack are eroded at an angle to the general coastal trend and beach areas in the vicinity of inlets, spits, and southwesterly ends of sections of coast are accreting.

Heavy minerals occurring in the beach sands take on more meaning as relates to spatial distribution when considered in the light of the foregoing dynamic factors. As will be demonstrated in succeeding paragraphs, heavy minerals appear to reflect these dynamic conditions.

Sediment Along the South Carolina Coast

Sediments comprising the beaches along the South Carolina coast were derived from the rotted crystalline rocks of the Piedmont region since Cretaceous time. This chemically resistant material, comprised essentially of quartz and minor heavy minerals, was carried by rivers and streams from the

Piedmont to the Atlantic Ocean where it was picked up by littoral currents and transported in the direction of predominant littoral current movement.

The sands along the northern South Carolina beaches to the Santee River tend to be largest in grain size, contain appreciable shell fragments, and generally contain less than 3 per cent heavy minerals. South from the Santee River to the Edisto River, the sand is smaller in average grain size and contains little shell while average heavy mineral content increases to 7 per cent. From Edisto River to the Georgia boundary, the sand is similar in grain size and shell content to that of the central South Carolina coast but heavy mineral content increases to 9 per cent.

As indicated above, the amount of heavy minerals disseminated in average beach sand varies over three distinct sections of coast with boundaries located at Santee and Edisto Rivers. The mineral species of the heavy mineral suite also vary in proportional amount for these sections of coast; the variation is caused by sediment contributions from rivers which drain different source areas.(2)

Between the Santee and Edisto Rivers, hornblende occurs in higher concentrations in the heavy mineral suite than in other sections of the coast. The geographic location and average per cent hornblende occurring in the heavy mineral fraction of average beach sand as determined from samples collected at two mile intervals near mean high water is listed below:

(1) From the North Carolina boundary to Santee River, hornblende averages 3 per cent of the heavy mineral fraction;
(2) Between Santee and Edisto Rivers, hornblende averages 12 per cent of the heavy mineral fraction with a maximum of 30 per cent occurring on Kiawah Island;
(3) South of Edisto River to the Georgia boundary, hornblende averages 4 per cent of the heavy mineral fraction.

The Santee River contributes sufficient hornblende that it may be considered a "point source" of this mineral and the predominant southwesterly littoral current moves the mineral to the southwest. Thus a "line source" of hornblende exists south from the Santee to the Edisto River. It will be shown that along this section of coast hornblende may be used as a mineral marker much the same as an irradiated glass particle of proper size and specific gravity has been used to follow the movement of quartz beach grains.

Concentrations of heavy minerals occur in some of the beaches as canoe-shaped surface deposits along the upper littoral zone and in the backshore area. These concentrations are sometimes present as thin beds or laminations at the berm of the beach. The largest natural concentrations of these "black sands" are situated along the central portion of the South Carolina coast and contain from 50 to 90 per cent heavy minerals. The concentrations vary in length from 1/4 to 3 miles, in width from 5 to 80 feet, and in depth from 3 inches to a maximum of 3 feet.(3) Surface concentrations occur only in areas of local erosion and thin beds or laminations in both areas of erosion or accretion. Concentration of the surface deposits in the eroding areas apparently results from the "panning" action of the waves in the upper littoral zone whereby littoral currents transport the lighter minerals leaving behind a residue of heavy minerals.

Since surface concentrates, thin beds and laminations of heavy minerals represent unique conditions, an evaluation of the general situation may be done best by considering the heavy minerals which are disseminated throughout the

beach sand. However, the ratio of the presence of a given mineral disseminated throughout the natural beach sand and in the surface concentrations may be used to show effects of dynamic factors. These different species in heavy mineral suite are listed in order of increasing specific gravity in the table of Fig. 3 for natural beach concentrates and average beach heavy mineral fractions. From this table, it is observed that the lightest of the heavy minerals are present in greater quantity in average beach sand than in natural heavy mineral concentrates. Hornblende, with the lowest specific gravity, generally averages 3 per cent in beach concentrates but averages 10 per cent in the heavy mineral fraction of average beach sand (Fig. 3). Tourmaline, epidote, kyanite, and staurolite (specific gravity range from 3.1 to 3.7) are present in greater amounts in the average beaches than in the concentrates. Garnet, holding a central position in the specific gravities, appears to remain constant. On the other hand, rutile leucoxene, ilmenite, zircon, and monazite of higher specific gravity, are present in greater quantities in natural concentrates than in average beach sand (Fig. 3). A similar correlation also exists in the dune system on the Isle of Palms.(4) Hornblende is present in greater quantity in the dunes where the heavy mineral concentration is lower.

Of all the heavy mineral species present along the central South Carolina coast, hornblende exhibits the greatest variation in relation to degree of heavy mineral concentration. High concentration of hornblende occurring in accreting areas and conversely the low concentrations of this mineral in eroding areas make it unique and diagnostic. Hornblende for this reason will be used as a marker mineral for considerations of applied energy and spatial relationships.

Application of Implied Method

The seaward approaches to Charleston Harbor illustrate the potential value of the foregoing method of analysis. Hornblende is used as a mineral indicator because of its physical properties, relative abundance, directional control from a "point source", and response to applied forces. Its general expression in relation to degree of concentration of heavy minerals (Fig. 3) and spatial distribution along the beaches (Fig. 4) is only meaningful if considered in the environmental setting summarized below:

(1) A near static sea level exists along the section of coast considered and predominant littoral currents are in a southwesterly direction.
(2) Erosion is in progress along Folly Island except for the northeastern end of the island and from the headland of Bull Island to the northeast end of Isle of Palms except in the vicinity of inlets.
(3) Accretion is in progress from the Isle of Palms to Charleston Harbor, northeastern Folly Island, Kiawah Island, and in the vicinity of inlets.
(4) Large surface concentrations of heavy minerals occur in the upper littoral zone and backshore area of the eroding portions of coast from Bull Island to the northeast end of the Isle of Palms. Elsewhere concentrations of heavy minerals are confined to thin beds or laminations in the backshore and beach area. Kiawah Island and the spit at the southwest end of Sullivans Island, both strong accretion areas, contain the least concentrations of heavy minerals.
(5) Departure from general coastal alignment occurs at the headland of Bull Island, in the vicinity of inlets, Sullivans Island, Cummings Point,

HEAVY MINERAL STATISTICS

Average Mineral Species in Heavy Fraction of Beach Concentrate and Average Beach Sand

Constituents	Beach Conc.	Average Beach	Specific Gravity
HORNBLENDE	3	10	3.0 - 3.5
TOURMALINE	1	2	3.1
EPIDOTE	12	20	3.3 - 3.4
KYANITE	2	3	3.6
STAUROLITE	5	6	3.7
GARNET	1	1	4.3
RUTILE	4	3	4.2-4.6
LEUCOXENE	4	3	4.6
ILMENITE	55	44	4.7
ZIRCON	10	6	4.7
MONAZITE	2	1	5.1
OTHER	1	1	-
TOTAL	100	100	-

1. INCLUDES SILLIMANITE, PYROXENE, HYPERSTENE, AND MAGNETITE

General Expression of Hornblende In Relation to Heavy Mineral Concentration

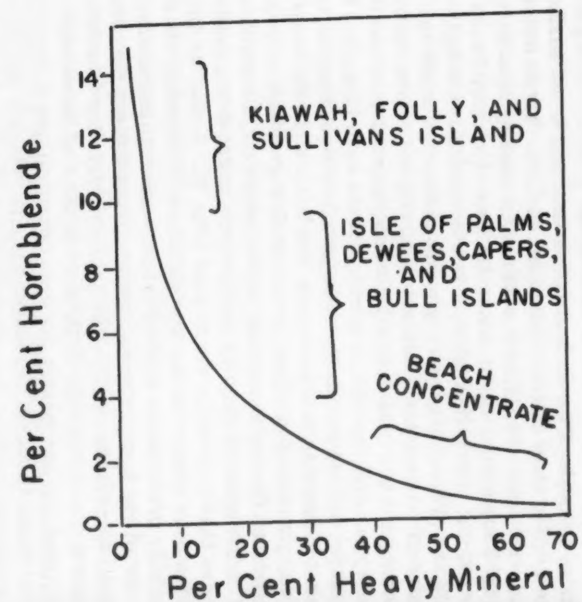


FIGURE 3

and Kiawah Island headland. Littoral currents may differ in these areas from general direction of littoral currents operating elsewhere along the coast.

The hornblende distribution appears to be inversely proportional to heavy mineral concentration along the section of coast considered as shown in the graph of Fig. 3. From Bull Island to Charleston Harbor, hornblende in natural heavy mineral concentration deposits ranges from 1 to 4 per cent and south of this point ranges from 5 to 12 per cent (Fig. 4). Heavy mineral concentrations are greatest for the lower values of hornblende in the vicinity of Bull, Capers, Dewees, and northeast Isle of Palms, while hornblende attains a maximum of 12 per cent in the heavy beds and laminations present in the beach berm portion of Kiawah Island (Fig. 4). This relationship is shown as "beach concentrates" in the graph of Fig. 3.

Hornblende in the disseminated heavy mineral fraction of average beach sand also differs in relative amounts depending on location, amount of heavy minerals, and coastal stability. The average beach sands of the Isle of Palms, Bull, Capers, and Dewees Islands contain between 6 and 30 per cent heavy minerals; the percentage of hornblende in the heavy minerals ranges from 2 to 9 per cent (Fig. 4). Kiawah and Sullivans Islands average about 4 per cent heavy minerals and hornblende in these fractions ranges from 11 to 25 per cent (Fig. 4).

The foregoing relationship of hornblende distribution in relation to heavy mineral concentration and geographic location is shown graphically in Fig. 3. The expression of per cent hornblende in relation to per cent heavy minerals is for the average grain size diameter of the heavy mineral fraction. This curve is the generalized average expression only since minor deviations do occur which might be resolved more adequately by more quantitative evaluation of individual sieve fractions.

Hornblende distribution shown in Figs. 3 and 4 are related to areas of erosion and accretion. In general it appears that hornblende is transported from areas of erosion and deposited in areas of accretion resulting in higher values of hornblende in zones of accretion. Being the lightest heavy mineral, hornblende is removed first and consequently as the heavy mineral concentration in eroding areas increases, the amount of hornblende in these areas decreases. Hornblende by virtue of its prismatic-elongate shape and therefore large surface area is also more easily transported than the other heavy minerals of greater specific gravity and more rounded shapes.

The distribution of hornblende disseminated through the beach sand along Folly Island suggests that the northeastern end of the island is a zone of accretion while the southwestern end is eroding. This conclusion has been confirmed by a comparative high water shoreline study based upon a report to Congress in 1935[5] and a recent map compiled from aerial photographs by the U. S. Coast and Geodetic Survey.

Littoral Drift into Charleston Harbor

The U. S. Corps of Army Engineers in a recent analysis of hornblende distribution in Charleston Harbor and the rivers draining into Charleston Harbor note an increase in this heavy mineral toward the harbor entrance (Fig. 5). Hornblende is present in the upreach areas of the rivers on the order of two to three per cent of the heavy mineral fraction but increases to as much as thirty per cent near the confluence of the Ashley and Cooper

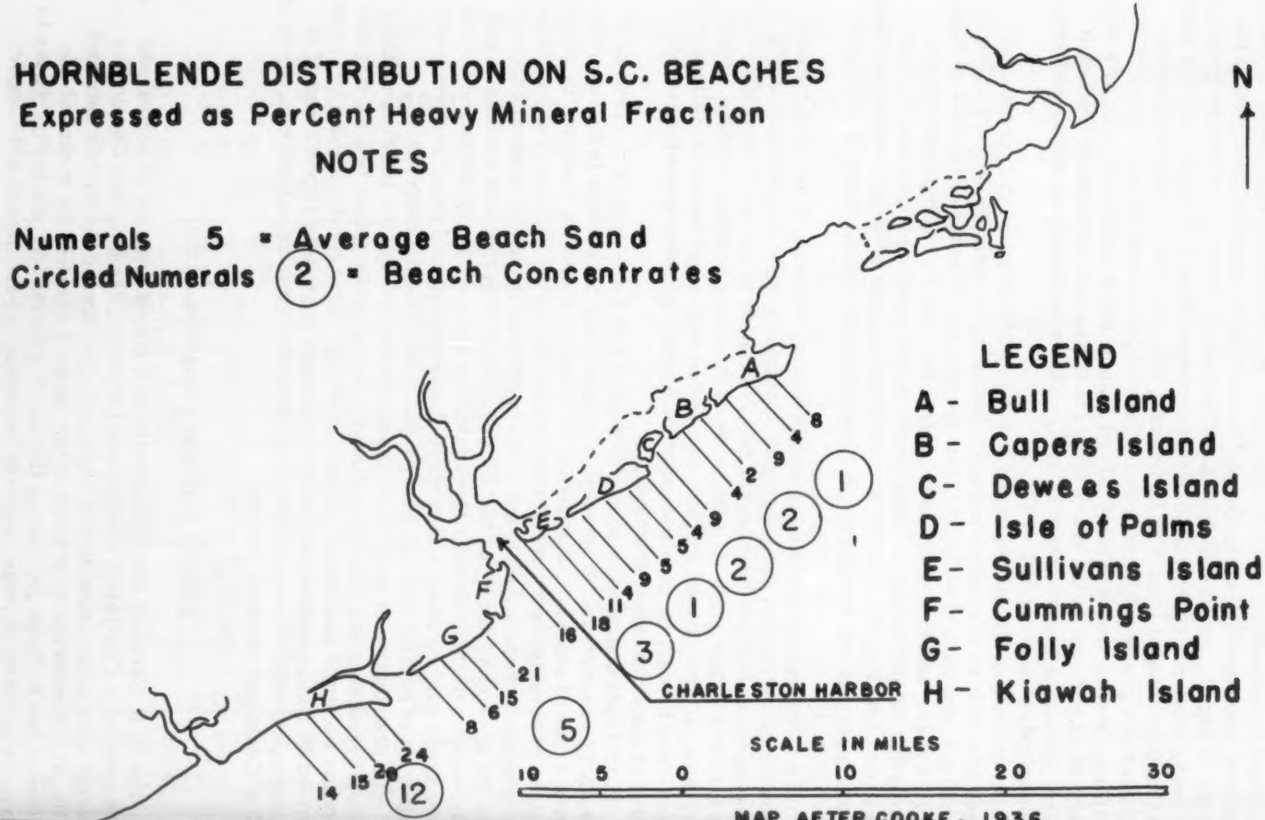
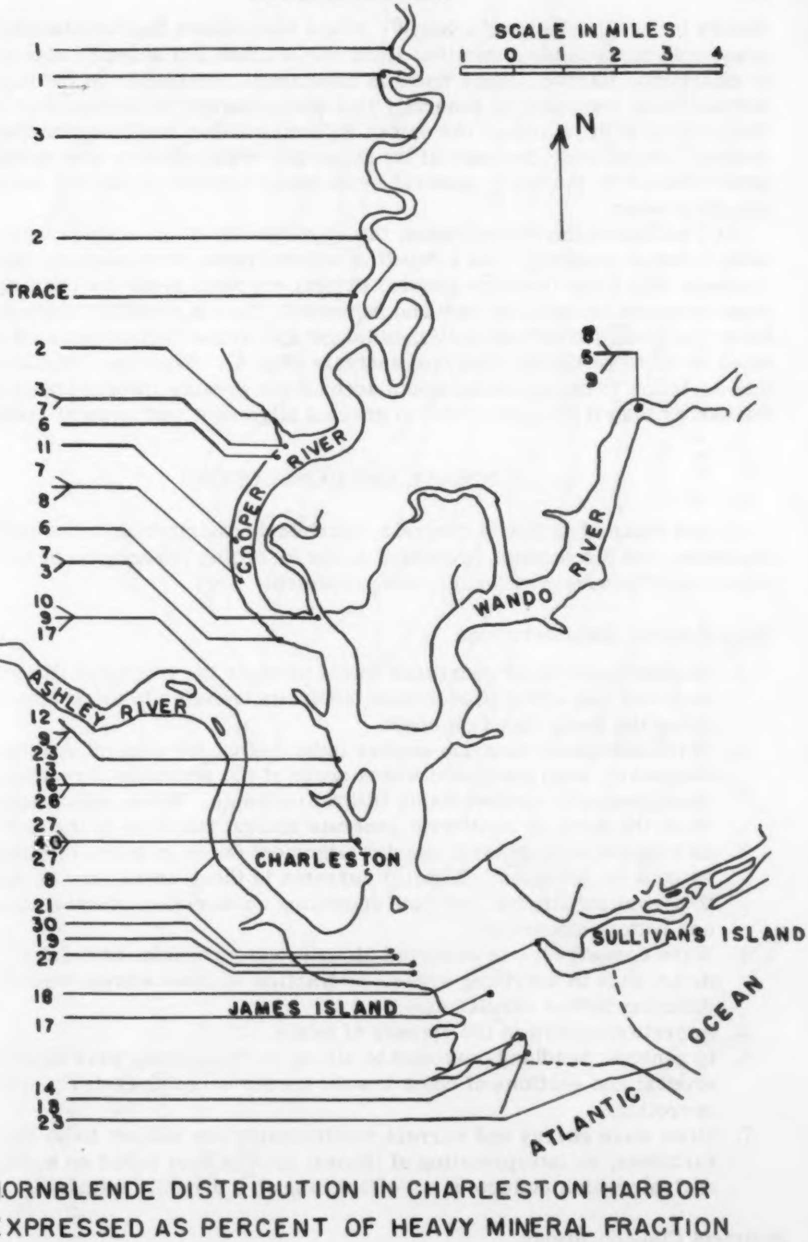

HORNBLENDE DISTRIBUTION ON S.C. BEACHES
Expressed as PerCent Heavy Mineral Fraction
NOTES
Numerals 5 = Average Beach Sand
Circled Numerals 2 = Beach Concentrates
N
LEGEND
A - Bull Island
B - Capers Island
C - Dewees Island
D - Isle of Palms
E - Sullivans Island
F - Cummings Point
G - Folly Island
H - Kiawah Island
CHARLESTON HARBOR
A
B
C
D
E
F
G
H
8
4
9
2
4
9
4
5
5
9
4
11
18
16
1
2
2
1
3
5
21
15
6
8
24
20
15
14
12
SCALE IN MILES
10 5 0 10 20 30
MAP AFTER COOKE, 1936

FIGURE 4



HORNBLENDE DISTRIBUTION IN CHARLESTON HARBOR
EXPRESSED AS PERCENT OF HEAVY MINERAL FRACTION

Material tested finer than 100 and coarser than 200 mesh sieve

CORPS OF ARMY ENGINEERS, 1950

FIGURE 5

Rivers in the Charleston Harbor.[6] It has been shown that hornblende is present in appreciable quantities along the beaches and seaward approaches to Charleston Harbor. Since there is substantial hornblende on the beaches and sediment transport is generally in a southwesterly direction, it is believed that littoral drift is around the spit of Sullivans Island and into Charleston Harbor. Hornblende, because of its shape and comparatively low specific gravity would be the heavy mineral most easily carried by littoral currents into the harbor.

At Charleston Harbor entrance, the shoreline is off at an angle from the main trend of coastline. As a result of this deviation from normal, counter currents may exist from the general littoral currents along the coast when the wind direction is from the east and southeast. Such a condition appears to exist at Cummings Point and Sullivans Island spit at the harbor entrance as indicated by wave refraction diagram analysis (Fig. 6). Expressed statistically, this deviation of the coastline would account for greater littoral transport into the harbor than if the spits were in general alignment with coastal trend.

SUMMARY AND CONCLUSIONS

On the basis of sediment analysis, hornblende distribution, wave refraction diagrams, and discussions presented in the foregoing paragraphs of this paper, the following conclusions are presented:

Hydrodynamic Considerations

1. Northeasterly wind generated waves produce the strongest littoral currents and effect predominant sediment transfer to the southwest along the South Carolina coast.
2. Wave refraction diagram studies indicate that for general coastal alignment, wind generated waves north of the southeast directions produce generally southwesterly littoral currents. Waves which approach from the south or southwest generate littoral currents to the northeast.
3. Deviations from general coastal alignment occur in areas of strong erosion or accretion. Littoral currents in these areas may be counter to prevailing littoral currents depending on direction of wave advance and bottom topography.
4. Wave convergence is observed along eroding portions of coast and wave divergence in accreting areas. Refraction of these waves results from differing bottom conditions.
5. Accretion occurs in the vicinity of inlets.
6. In general, headlands exposed to strong northeasterly wave attack are eroding and sections of coast toward the lee or southwesterly side are accreting.
7. Since wave energy and current relationships are subject to so many variables, an interpretation of littoral drift is best based on sediment analysis which reflects a net effect in such a dynamic situation.

Sediment Considerations

1. Heavy mineral concentrations occurring in the upper littoral zone and backshore areas are located in eroding areas while thin beds or laminations of heavy minerals occur in the beach berm of eroding and accreting areas.

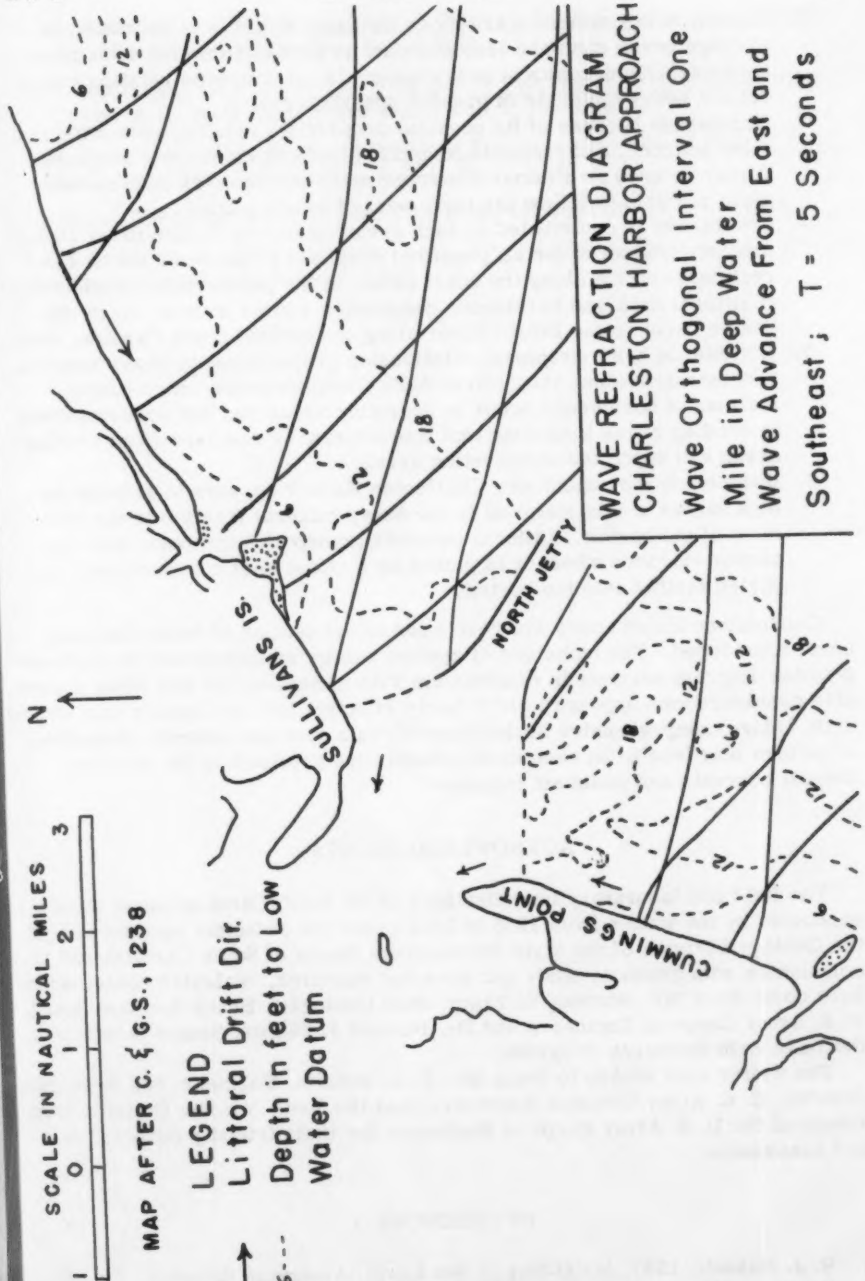

WAVE REFRACTION DIAGRAM
CHARLESTON HARBOR APPROACH
Wave Orthogonal Interval One
Mile in Deep Water
Wave Advance From East and
Southeast ; T = 5 Seconds
SCALE IN NAUTICAL MILES
1 0 1 2 3
MAP AFTER C. & G.S., 1238
LEGEND
Littoral Drift Dir.
Depth in feet to Low
Water Datum
N
SULLIVANS IS.
NORTH JETTY
CUMMINGS POINT

FIGURE 6

2. Lighter heavy minerals are more abundant in areas of accretion or average beach sand and less abundant in areas of erosion or in natural concentration deposits of heavy minerals. A converse relation exists for the heavy minerals of greater specific gravity.
3. Hornblende because of its physical properties, relative abundance in a "line source", and response to applied force is unique and diagnostic. It may be used as a mineral marker much the same as an irradiated glass particle to follow the movement of quartz grains.
4. Hornblende is contributed in such great volume by Santee River that it may be considered for all practical purposes a "point source". Directional control along the coast exists in the predominant southwesterly littoral drift and hornblende constitutes a "line source" from the Santee River to the Edisto River along the central South Carolina coast.
5. Hornblende has a graphical relationship proportional to heavy mineral concentration along the central South Carolina coast. High concentrations of hornblende occur in accreting areas and low concentrations in eroding areas indicating that this mineral is removed from eroding areas and deposited in accreting areas.
6. Hornblende movement into Charleston Harbor appears to account for high values of this material in the heavy mineral fraction in the sediment of the harbor. Littoral currents transport hornblende into the harbor entrance where it is picked up by tidal inlet currents and transported farther into the harbor.

Conclusions listed above are restricted to the section of South Carolina coast considered. The technique of applied energy as interpreted by wave refraction diagram analysis in combination with mineralogical and other observable considerations appears to be a fairly effective tool for inquiry into littoral drift. More comprehensive evaluations of grain size and other combinations of factors may lead to an even more quantitative approach to the study of littoral currents and sediment transport.

ACKNOWLEDGEMENTS

The field and laboratory investigations of the South Carolina coast were conducted by the writer from 1956 to 1958 under the principle sponsorship of the Geology Division of the State Development Board of South Carolina and in conjunction with graduate study and personal research. Valuable contributions have come from Mr. Norman E. Taney, Staff Geologist, Beach Erosion Board, U. S. Army Corps of Engineers and Dr. Howard J. Pincus, Senior Scientist of the Lake Erie Research Program.

The writer also wishes to thank Mr. E. A. Schultz, Engineer, San Francisco District, U. S. Army Corps of Engineers, and the South Atlantic Division Laboratory of the U. S. Army Corps of Engineers for their friendly cooperation and assistance.

REFERENCES

1. R. J. Russell, 1957, Instability of Sea Level, American Scientist, Vol. 45, No. 5, p. 428.

2. J. Neiheisel, 1957, Reconnaissance Study of South Carolina Beach Sands, unpublished S. C. State Dev. Board Bull.

3. J. Neiheisel, 1958, Heavy Mineral Beach Placers of the South Carolina Coast, S. C. State Dev. Board Bull., Vol. 2, No. 1, p. 1.

4. J. Neiheisel, 1958, Origin of the Dune System on the Isle of Palms, S. C., S. C. State Dev. Board Bull., Vol. 2, No. 7, p. 47.

5. House Document No. 156, 74th Congress, 1st Session, 17 April 1935.

6. U. S. Corps of Army Engineers Lab., South Atlantic, Petrographic Studies, Charleston Harbor Project, 1951.

P

La
waves
dock
and t
ple n
ment
comp

Li
dock
as d
enco
these
the a
drill
of th
freig
duce
tic s

Note
a
i
i

1. A
E
2. C
E
3. J
C

Journal of the

WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

MODEL STUDY OF MOORING FORCES OF DOCKED SHIP

R. L. Wiegel,[1] R. A. Dilley,[2] and J. B. Williams[3]

SUMMARY

Laboratory data are presented on the forces induced by water gravity waves in the camels and mooring lines of a Liberty Ship moored alongside a dock for waves up to five feet in height and with periods up to one hundred and twenty seconds (prototype dimensions). It was not possible to obtain simple natural periods of surge, sway, etc. for this particular mooring arrangement as one motion induced another. The motion of the moored ship was very complicated for all of the test conditions.

INTRODUCTION

Little is known of the forces acting on the camels and mooring lines of a docked ship which is subject to wave action. The theory of mooring forces as developed to date(4,9) is for a much simpler set of conditions than actually encountered. Very few prototype measurements have been made,(1,2,3) and these have been for the case where the predominant forcing phenomena. With the advent of offshore moorings for super-tankers, and moored offshore drilling rigs it is necessary for the engineer to have a better understanding of the phenomenon. In addition, there are many areas of the world where freighters go that do not have well protected harbors and where the wave induced forces are a serious problem. It is generally understood that the elastic spring system (the mooring line and the camels) is non-linear and the

Note: Discussion open until November 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2071 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. WW 2, June, 1959.

1. Associate Research Engr., Inst. of Eng. Research, Univ of California, Berkeley, Calif.

2. Graduate Research Engr., Inst. of Eng. Research., Univ. of California, Berkeley, Calif.

3. Jr. Engr., Inst. of Engr. Research, Univ. of California, Berkeley, California.

damping forces are non-linear. In addition, the forcing function is non-linear, depending to a marked degree under certain circumstances upon the second-order effect of wave mass transport.

It was believed that a series of model studies was necessary to obtain an insight into the characteristics of mooring forces as a function of wave height and wave period. A study in the laboratory allows the systematic variation of the various parameters which is not possible in nature. Froude's modeling law was used, with the mooring lines being modeled correctly with respect to both weight distribution and elasticity. Details of the analysis of the modeling problem will not be presented herein as they have been given elsewhere.(7) One critical test of the validity of the proper modeling of the mooring system is to compare the natural period of surge (longitudinal oscillation of the ship) of a model with the prototype as this vibration is due to the mooring system. This has been done for one floating structure, a drydock, and has been found to be satisfactory.(3) It is reasonable, then, to expect the model study to predict at least the general characteristics of the prototype.

It was believed that a model of a freighter tied up alongside a dock equipped with camels to protect the dock would be of most general interest. The results of such a test are presented herein. The model was of an ARG-11 (a Liberty Ship hull) as shown in Fig. 1 and 3. A geometrically similar model was furnished by the David Taylor Model Basin. From dimensions provided by the U. S. Naval Civil Engineering Laboratory, Port Hueneme, California (Fig. 1) and measurements on the geometrically similar model, the length scale ratio was determined to be 1:86. This established the force ratio at 1:620,000 and the time scale ratio at 1:9.27 according to Froude modeling principles.(7) The model was made dynamically similar by the methods described by Wiegel, Clough, Dilley and Williams.(7) The force meters were designed using the same principle described in Ref. 7.

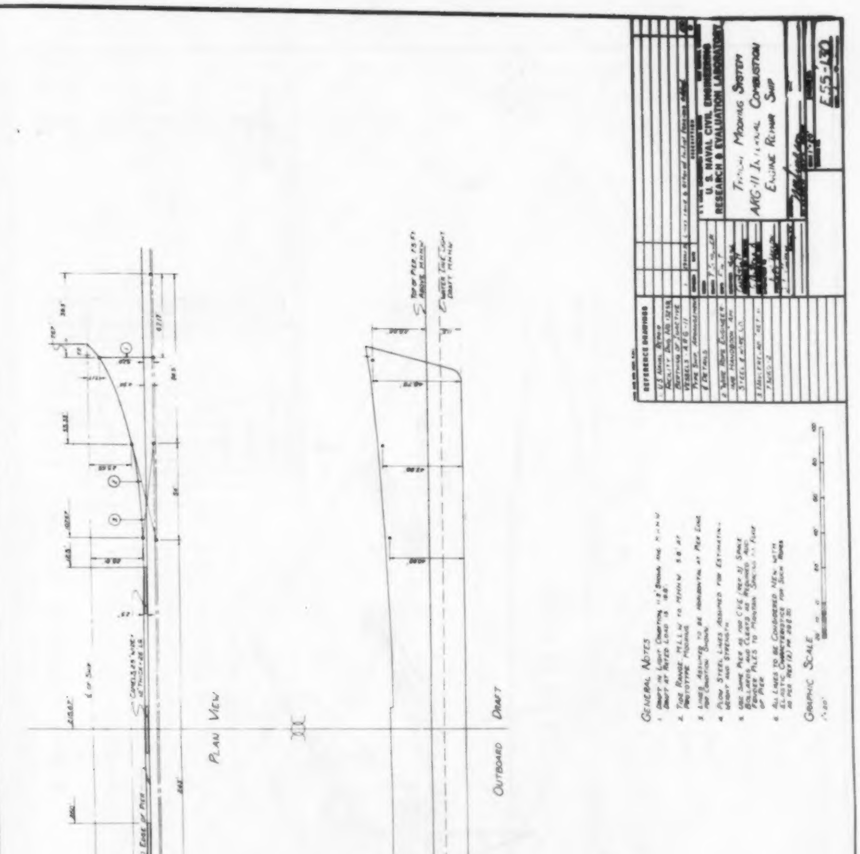
Considering the limited amount of information available to the design engineer on this problem, it is believed that the results presented herein should be useful to him. It is cautioned, however, that these data should not be used as a guide for ships which differ by much from the Liberty Ship or for a mooring arrangement that differs by much from the moorings used in these tests. Model studies would be needed for the specific ship and set-up.

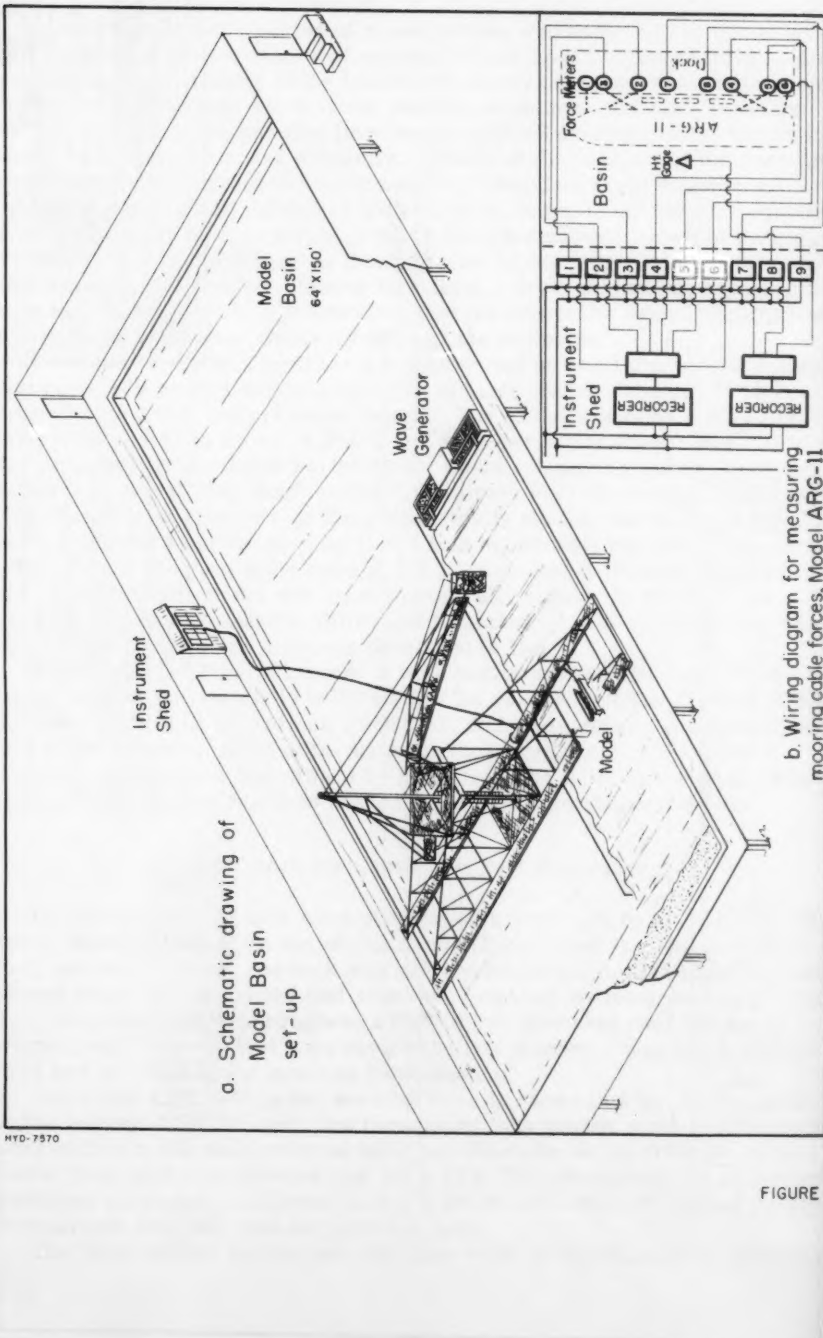
Laboratory Equipment and Test Procedure

The tests on the ARG-11 were performed in the 2-1/2 ft. by 64 ft. by 150 ft. Model Basin at the University of California (Figs. 2 and 4). The immediate area around the model and dock was surrounded by a 2 ft. high plywood windscreen which was supported just above the height of the maximum wave (Fig. 4a). When tests were in progress a canvas was stretched over the top of the windscreen. With the test area covered in this manner it was found that the wind had no effect on the mooring cable forces.

The model dock used in the mooring tests is shown in Figs. 3, 4a, and 5. In the interest of expediency, and because no information was available on the dock stiffness, the dock stiffness used was the same as the stiffness of the model dock used in a previous test (on a CVE-78). This made the equivalent prototype dock have a stiffness of 41.3 × 106 lb.-in., which was about twenty-five percent less stiff than the previous dock.

The force meters for the mooring lines were designed using the procedure


PLAN VIEW
OUTBOARD DRAFT
GENERAL NOTES
GRAPHIC SCALE
U. S. NAVAL CIVIL ENGINEERING RESEARCH & EVALUATION LABORATORY
TYPICAL MOORING SYSTEM
ARG-11 INTERNAL COMBUSTION ENGINE REPAIR SHIP
E55-130

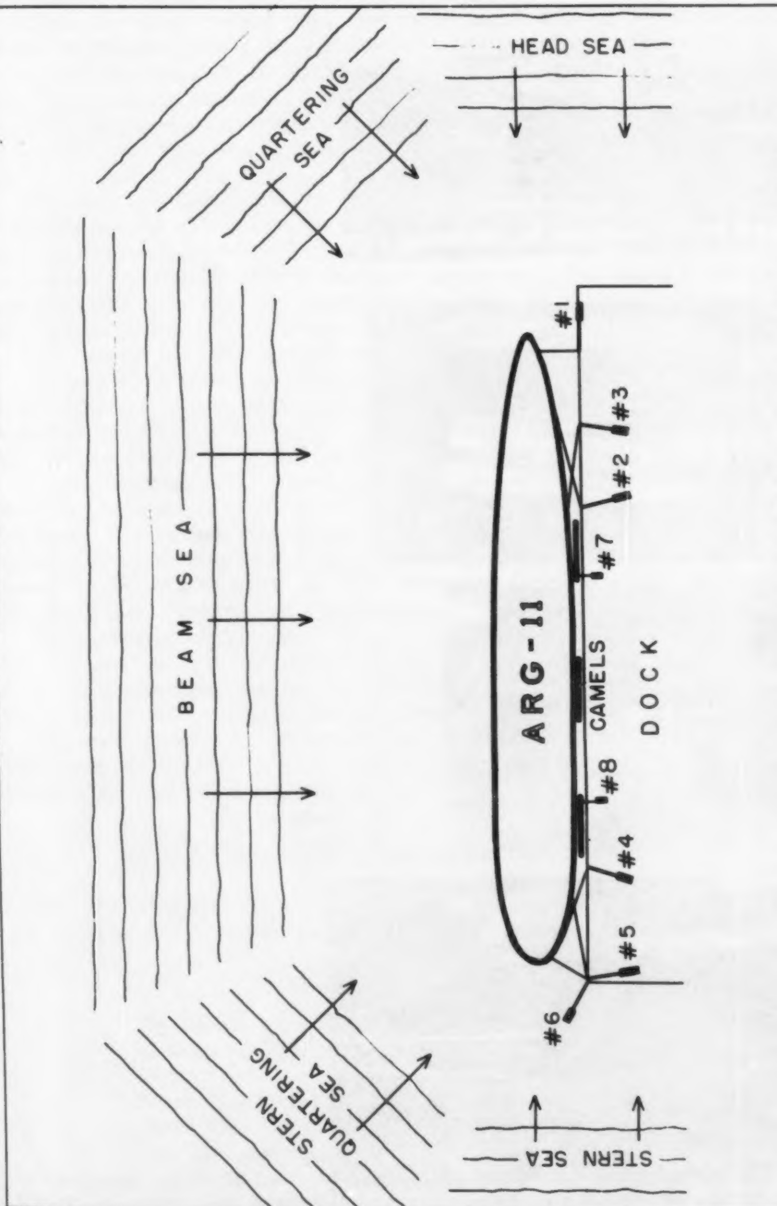


a. Schematic drawing of Model Basin set-up

b. Wiring diagram for measuring mooring cable forces, Model ARG-11

HYD-7570

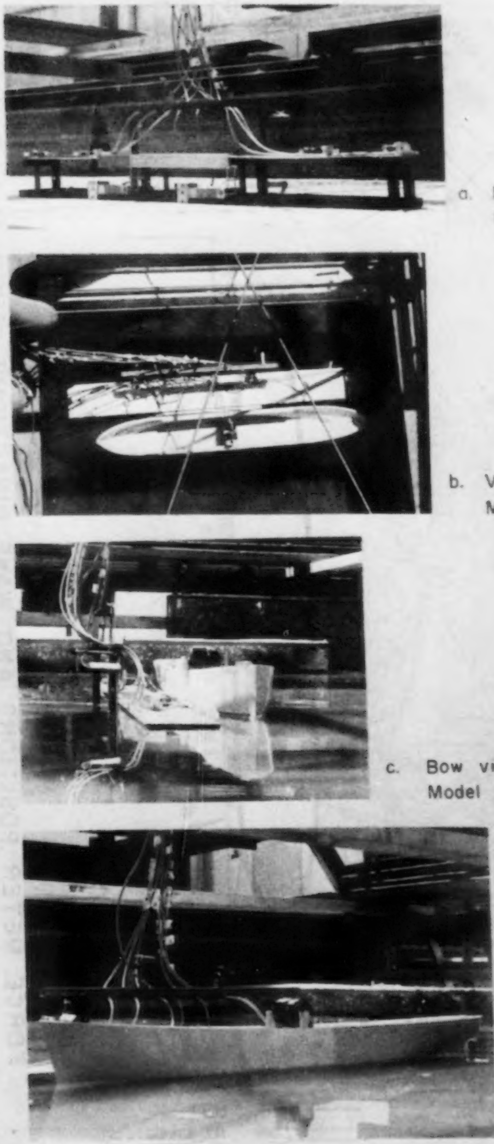
FIGURE 2



FORCE METER POSITIONS AND WAVE DIRECTIONS, ARG-II Model Tests

HYD-7594

FIGURE 3



a. Dock for Model ARG-II.

b. Vertical view of Model ARG-II set-up.

c. Bow view of Model ARG-II set-up.

d. Side view of Model ARG-II set-up.

HYD-7942

FIGURE 4

described by Wiegel, Clough, Dilley and Williams.[7] The dock stiffness was built into the bumper which the camel rested against using two force meters of the cantilever type.[8] This enabled the horizontal point of reaction to be found if it was desired. Since the vertical point of reaction would vary as the ship moved during the tests, it was necessary to use some type of device so that this variance would not affect the force-meter calibration or the stiffness of the system. This was accomplished with the device shown in Fig. 5 (Detail "A").

The dock was fastened to the basin floor with two screws which held it to Phillip's sheilds driven into the floor of the model basin. One screw was used as a pivot point when the model heading was changed and the other screw was used to clamp the dock in the desired position. The layout of the model and dock with mooring lines denoted by numbers from 1 - 6 is shown in Fig. 3. Force meters Nos. 7 and 8 were used for measuring the compression forces in the "camel" (See wiring diagram, Fig. 2).

A tough black string was used to model the prototype wire-rope mooring lines. The stiffnesses of the model lines were found experimentally and the force meters were adjusted as needed to provide the additional stiffness in the various lines. Details of the force meters are shown in Fig. 5.

All the mooring-line forces, the two compression-meter forces and the surface time history (wave record) were recorded on Brush Electric Co. recorders. The surface-time history was measured by a parallel-wire resistance-type meter.[6] The waves were generated by a piston-type wave generator.[5] Waves were absorbed downwave from the model by means of stainless-steel borings and a sand beach.

The model was made ready for a test by placing it in the desired orientation and covering the test area with canvas. Attempts were made to determine the various natural periods of oscillation, but the motions damped out very rapidly and one motion induced another.

The wave generator was started and the surface-time history and all forces were recorded for ten or more cycles. The water in the model basin was allowed to become calm and then the next wave condition was used.

Experimental Results and Discussion

The following table gives a list of the data presented in Appendix I. Runs not included were used for preliminary test purposes.

Table A. Test Conditions

Head Seas	Runs 1-27
Quartering Seas	Runs 28-55
Beam Seas	Runs 56-83
Stern Quartering Seas	Runs 84-111
Stern Seas	Runs 112-138

In analyzing the force records the four highest peaks during a run of 5 to 25 waves were averaged to give the average force on a meter during the run. All values are presented in terms of the prototype.

In addition to the tabulation of data, graphs showing the relationship between force divided by wave height and wave period are shown in Fig. 9. From these one can get a general idea of the value of the natural periods of motions

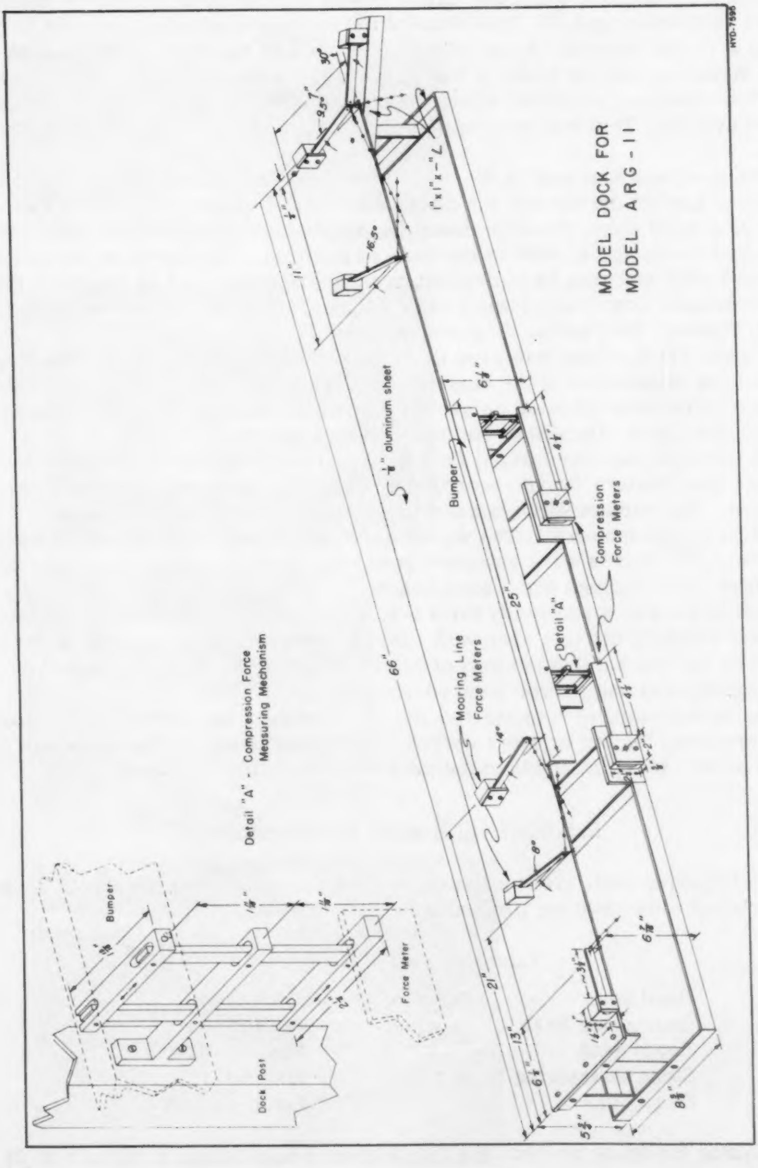
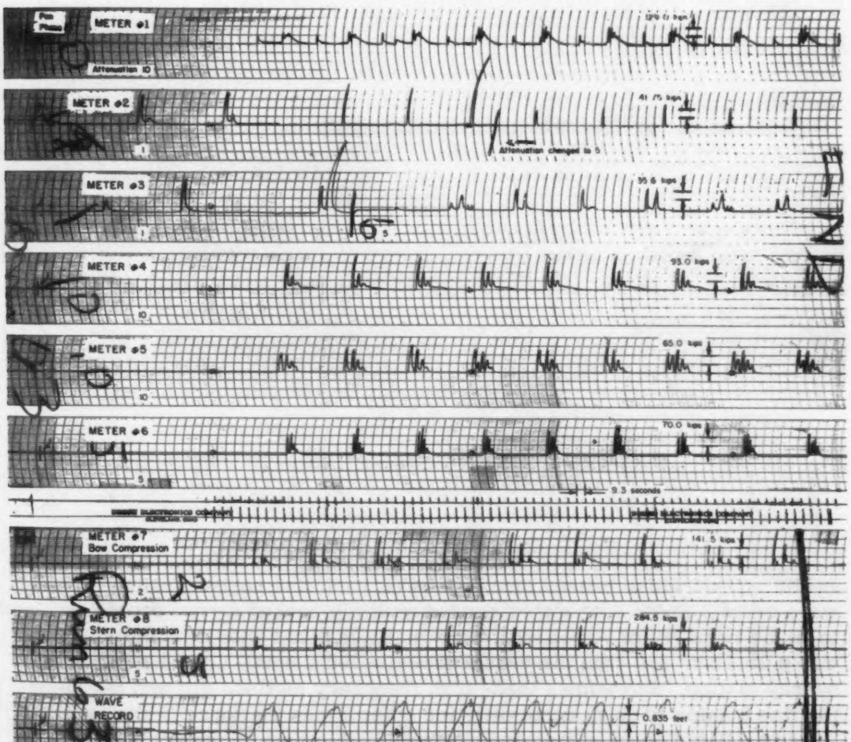

MODEL DOCK FOR
MODEL ARG-11
Detail "A" - Compression Force
Measuring Mechanism
Bumper
Force Meter
Dock Post
aluminum sheet
Compression
Force Meters
Mooring Line
Force Meters
Detail "A"
66"
25"
21"
11"
13"
1"x1"x-"L

FIGURE 5

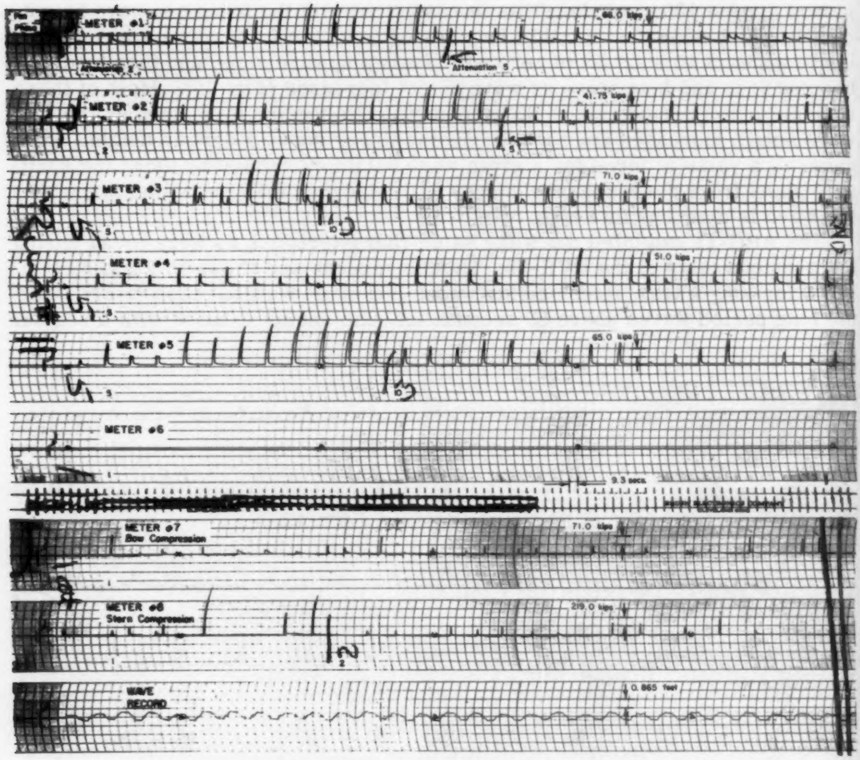
of the vessel. For some conditions these curves show a high point for a wave period around 100 seconds. This is consistent with the force vs. wave height curves which show that for this long period wave the force versus wave height curve appears to start from zero with a steep upward slope and then tends to become less and less steep as wave height increases (Fig. 10). Most of these curves for the periods above 60 seconds tend to become less steep as the wave height increases and tend to become linear above a certain height. In general, a value of force for a given wave height calculated by using the limiting values from the resonance curves will be higher than the actual force if the wave is over one foot high.

The ARG-11 exhibited much the same characteristics as were found during the previous tests of the CVE-78. Sample records are shown in Figs. 6, 7 and 8 for long waves, intermediate waves and short waves, respectively. For the long period waves above 50 seconds the ship seems to move with a regular



SAMPLE RECORD, RUN 63
MODEL ARG - 11

FIGURE 6

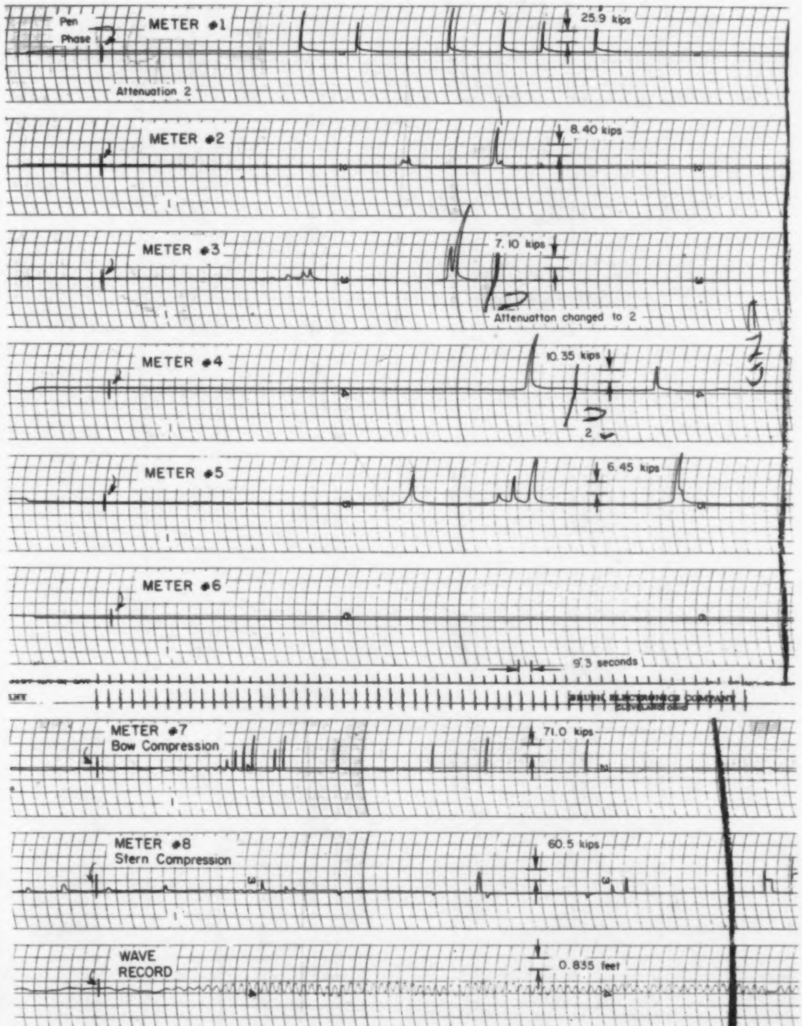


SAMPLE RECORD, RUN 11
MODEL ARG-11

FIGURE 7

cyclic motion which caused force peaks on each line corresponding to each wave crest. These long waves caused multiple peaked force records referred to in the remarks of Appendix I. The multiple peaks indicated that the line was pulled taut and then the vessel experienced a small amplitude high frequency motion. Sometimes these high frequency motions allowed the mooring line tension to go to zero while for other times the force in the mooring lines oscillated about some positive force. This high frequency motion was observed to be primarily roll for vessel headings other than head and stern seas these multiple peaks may have been caused by pitching, small surging, or even rolling. In general the multiple peaks occurred for the long, high waves.

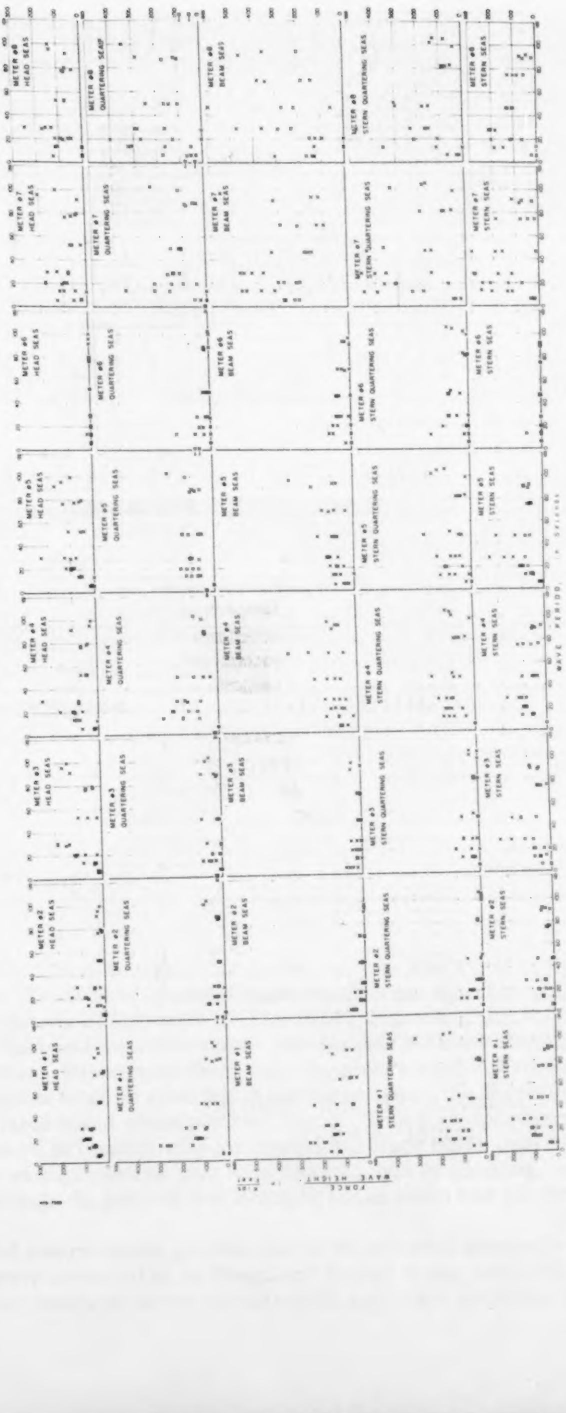
Waves of intermediate periods (20 to 50 seconds) generally caused forces for wach wave referred to as "regular" forces in the remarks. However, these forces exhibited large variations in amplitude from one wave to the



SAMPLE RECORD, RUN 80
MODEL ARG-11

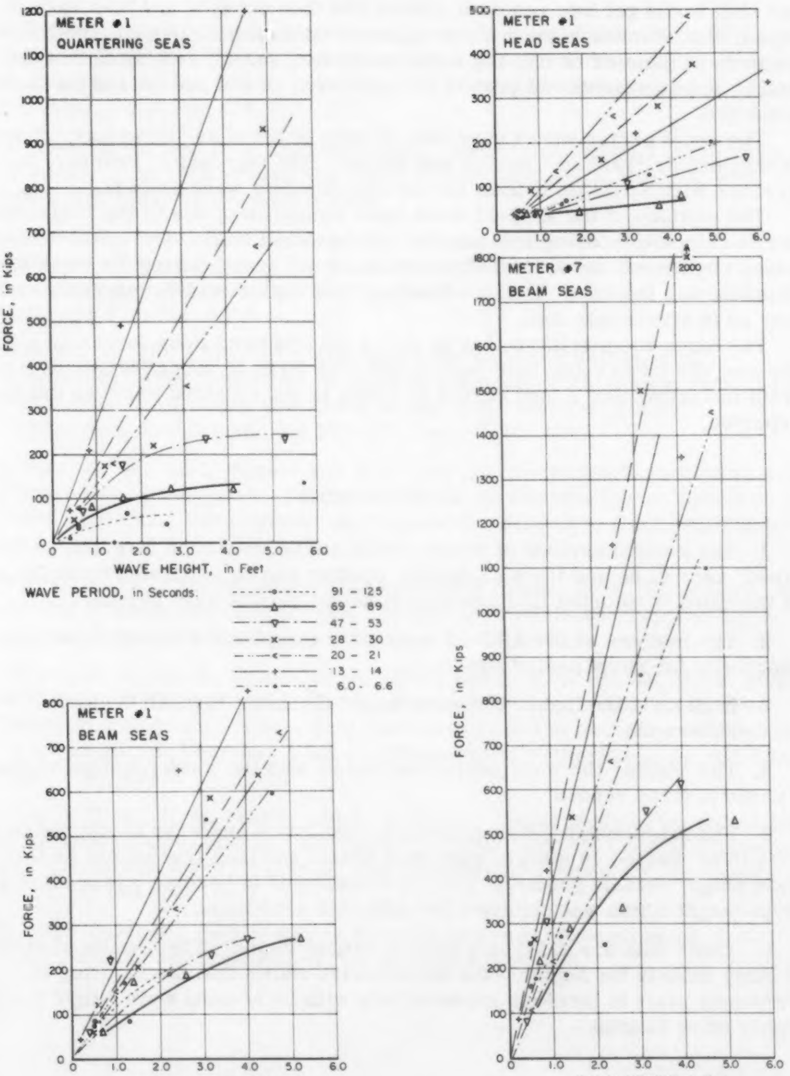
HYD-7598

FIGURE 8



RATIO OF MOORING CABLE FORCES TO WAVE HEIGHT AS A FUNCTION OF WAVE PERIOD. MODEL ARG-11

FIGURE 9



EXAMPLES OF MOORING LINE FORCES *VERSUS* WAVE HEIGHT AND WAVE PERIOD, MODEL ARG-11

HYD-7500

FIGURE 10

next. This condition was noted as a "beat" in the remarks. It appeared that the ship would get into resonant motion and then out of it and then back in again, etc. For the highest waves at these intermediate periods the force records became more like the records for long waves, indicating that the height plays an important part in the regularity of the motion and the forces produced.

The short period waves gave rise to what is noted as "irregular," "very irregular" or "random" motion and forces. The regularity, however, increases with height as it does for the intermediate wave periods.

The motions of the ARG-11 were quite complicated due to the restraints put on it by the mooring system. In both head and stern seas meter #6 hardly every registered any force. Observation of the model during the tests showed that this was the case. In these headings the vessel seldom moved in such a way as to stress this line.

The force records show that in any of the test conditions forces as high as the average force could be expected after the first or second waves of a wave train indicating that a long series of waves is not required to cause the forces reported.

CONCLUSIONS

1. The natural periods of motion could not be measured directly. A "combined" natural period for a particular heading can be estimated from the plots of the ratio of mooring line force wave height versus wave period.

2. The motions of the ARG-11 were very complicated for all conditions, especially for short period waves.

3. In general the higher the wave height the more regular the force records became.

4. The shorter the wave period the higher was the wave required to cause a uniform force record.

5. The relationship between mooring-line force and wave height for some conditions started up steeply with wave height and then leveled out as the wave height became greater. The curves seemed to become linear after a given height which was different for different conditions.

6. These data are valid only for the ranges tested. The results of studies of other models for higher wave heights have shown that the mooring line force may start to increase exponentially with increasing wave height for the higher wave heights.

ACKNOWLEDGMENTS

The authors wish to express their appreciation to A. L. Arnold, S. F. Whisenand, and J. Kukk for assisting with the laboratory work and in helping reduce the data, and to M. M. Lincoln for preparing the illustrations. The tests were conducted under contract to U. S. Navy, Bureau of Yards & Docks, U. S. Naval Civil Engineering Laboratory.

REFERENCES

1. O'Brien, J. T., Forces on Moored ships due to wave action, Proc. First Conf. on Ships and Waves, Council on Wave Research, The Engineering Foundation, and the Soc. of Naval Architects and Marine Engineers, Berkeley, Calif., pp 455-473, 1955.

2. O'Brien, J. T., and D. I. Kuchenreuther, Forces induced on a large vessel by surge, Jour., Waterways and Harbors Division, ASCE, vol. 84, ww2, Paper No. 1571, 29 pp, March 1958.

3. O'Brien, J. T. and D. I. Kuchenreuther, Free Oscillations in surge and sway of a moored floating dry dock, Proc. Sixth Conf. on Coastal Engineering, Council on Wave Research, The Engineering Foundation, Berkeley, Calif., pp 878-894, 1958.

4. Sommet, J., The Motion of a ship under the action of seiche, Proc: Symposium on the Behaviour of Ships in a Seaway, 25th Anniversary of the Netherlands Ship Model Basin, Wageningen, H. Veenman & Zonen, Wageningen, Netherlands, pp 354-373, Sept. 7-10, 1957.

5. Snyder, C. M., R. L. Wiegel and K. J. Bermel, Facilities for studying water gravity wave phenomena. Proc. Sixth Conf. on Coastal Eng., Council on Wave Research, The Engineering Foundation, Berkeley, Calif., pp 231-251, 1958.

6. Wiegel, R. L., Parallel wire resistance wave meter, Proc. First Conf. on Coastal Engineering Instruments, Council on Wave Research, The Engineering Foundation, Berkeley, Calif., pp 39-43, 1956.

7. Wiegel, R. L., R. W. Clough, R. A. Dilley and J. B. Williams, Model study of floating drydock mooring forces, Proc. of the Symposium on the Behavior of Ships in a Seaway, 25th Anniversary of the Netherlands Ship Model Basin, H. Veenman & Zonen, Wageningen, Netherlands, pp 563-583, Sept. 7-10, 1957.

8. Wiegel, R. L., K. E. Beebe, and R. A. Dilley, Model study of the dynamics of an LSM moored in ocean waves, Proc. Sixth Conf. on Coastal Engineering, Council on Wave Research, The Engineering Foundation, Berkeley, Calif., pp 844-877, 1958.

9. Wilson, Basil W., Ship response to range action in harbor basins, Proc. ASCE, vo. 76, Separate No. 41, 29 pp, November 1950.

TABLE I: MOORING FORCES FOR A MODEL ARG-11 IN HEAD SEAS

Run No.	Wave Height ft.	Wave Period sec.	Force in Meter - Kips 1	2	3	4	5	6	7	8	Remarks
1	0.5	92	30	16	70	60	90	0	17	33	Regular motion - periodic forces on all but #6 approxi-
2	0.9	96	35	23	155	27	125	0	185	120	mately at wave period. #1, 3, 4 are in phase and 2
3	1.6	100	67	29	235	41	155	0	205	225	and 5 are in phase. Multiple peaks on mooring line
4	0.7	77	37	18	44	49	90	0	58	48	records. Compression forces are quite irregular.
5	1.9	78	43	17	120	135	155	0	115	150	do
6	3.7	79	55	25	120	165	225	0	150	135	do
7	4.2	80	78	67	160	195	240	0	165	325	do
8	1.0	53	41	27	99	73	110	0	21	105	do
9	3.0	53	105	52	175	155	170	0	175	210	do
10	5.7	53	165	77	98	260	225	0	370	385	do
11	0.8	30	91	96	150	190	175	0	130	190	Tension records show beating - 1, 3 and 4 in phase -
12	2.4	30	160	52	460	210	300	0	190	385	2 - 5 in phase - compression forces fairly regular -
13	3.7	29	280	175	570	315	375	0	290	640	some multiple peaks.
14	4.5	30	375	200	640	385	335	0	565	580	do
15	0.5	21	45	77	38	36	44	0	91	23	Irregular records for all meters.
16	0.6	21	46	5	12	64	68	0	47	35	do
17	1.4	21	135	110	145	230	125	0	240	155	Regular forces - both tension and compression -
18	2.7	21	260	235	330	285	275	0	260	240	beating on all meters - some multiple peaks.
19	4.4	20	485	310	505	445	435	0	460	350	do
20	0.4	13	37	3	14	3	0	0	0	0	Irregular force records - not at wave period - beats.
21	0.9	14	46	18	68	68	56	0	43	0	do
22	3.2	14	220	245	80	345	175	4	315	345	do
23	6.2	14	335	230	190	570	325	105	705	730	Forces regular at wave period - beats.
24	0.6	6.0	30	7	11	45	12	0	81	80	Very irregular forces - some high peaks in each line -
25	1.5	6.3	61	0	18	27	2	0	81	0	no regularity to the motion.
26	3.5	6.6	125	0	44	44	27	0	87	0	do
27	4.9	6.6	200	34	80	90	48	0	115	0	do

TABLE II: MOORING FORCES FOR A MODEL ARG-11 IN QUARTERING SEAS

TABLE II: MOORING FORCES FOR A MODEL ARG-11 IN QUARTERING SEAS

Run No.	Wave Height ft.	Wave Period sec.	Force in Meter - Kips 1	2	3	4	5	6	7	8	Remarks
28	0.6	100	32	52	41	40	72	0	71	205	Forces regular at wave period - multiple peaks; 3
29	0.6	103	40	47	31	63	125	1	145	270	and 4 generally in phase; 2 and 5 generally in phase.
30	1.7	125	65	44	33	60	120	12	125	135	do
31	2.6	125	42	54	34	82	130	16	90	98	do
32	0.9	89	81	29	73	48	78	0	43	245	Regular forces on all meters - multiple peaks - beats.
33	1.6	87	100	42	31	82	155	27	130	255	do
34	2.7	88	120	60	48	155	215	71	235	265	do
35	4.1	89	120	91	130	215	240	80	155	485	do
36	0.7	50	69	20	62	130	99	0	78	175	Forces at wave period on all meters - some beating;
37	1.6	50	175	37	6	160	170	11	235	250	multiple peaks.
38	3.5	50	235	84	66	220	240	71	410	495	do
39	5.3	50	235	99	83	175	280	155	560	525	do
40	0.5	28	52	1	1	70	33	0	85	0	Regular forces at wave period on #1 - others not
41	1.2	29	175	34	5	220	125	27	135	340	regular.
42	2.3	29	220	110	7	335	220	27	330	375	Regular forces at wave period on all meters -
43	4.9	29	935	145	320	370	525	285	735	1450	multiple peaks - beats.
44	0.6	20	77	0	3	56	41	0	26	0	Irregular forces - not at wave period.
45	1.4	20	180	67	11	275	130	0	310	170	All forces at wave period - beats.
46	3.1	20	355	94	135	605	450	100	470	1250	do
47	4.7	20	670	170	200	585	545	235	455	1200	do
48	0.4	14	72	16	35	68	42	0	0	21	Only occasional forces on all meters - very irregular
49	0.8	14	210	59	22	68	47	56	135	135	motion indicated.
50	1.6	14	490	125	76	395	200	51	575	495	#1 regular at wave period with beating - others ir-
51	4.5	14	1200	320	255	695	320	690	1000	1450	regular.
52	0.4	6.5	10	0	0	0	0	0	0	68	Very random motion - #8 is fairly regular at wave
53	1.4	6.6	0	0	0	0	0	0	0	36	period.
54	3.9	6.6	0	0	29	0	0	0	94	180	Compression forces are generally regular at wave
55	5.7	6.6	136	0	0	0	0	0	270	365	period - others random.

TABLE III: MOORING FORCES FOR A MODEL ARG-11 IN BEAM SEA

Run No.	Wave Height ft.	Wave Period sec.	Force in Meter - Kips 1	2	3	4	5	6	7	8	Remarks
56	0.5	91	75	1	22	54	92	0	160	225	Regular forces at wave period - some beats - some
57	0.5	97	84	0	18	60	125	3	255	240	multiple peaks. #1 has a peak about one-half the
58	1.3	94	85	0	0	120	115	39	185	440	maximum between each wave.
59	2.2	94	190	0	0	170	160	45	255	420	do
60	0.7	70	59	1	1	155	100	2	215	275	Regular forces at wave period on all meters. #1 has
61	1.4	69	170	1	0	415	150	31	290	460	more than one peak per wave; some multiple peaks.
62	2.6	70	185	48	33	315	215	140	335	470	do
63	5.2	70	268	94	83	310	185	200	530	735	do
64	0.4	47	61	9	0	82	42	0	80	23	Irregular forces.
65	0.9	47	220	0	0	125	145	46	305	540	Irregular forces.
66	3.2	47	230	38	33	325	280	195	555	865	All meters but 2 and 3 record regular force trace.
67	4.0	47	265	48	62	300	285	255	615	870	do
68	0.6	29	120	4	0	91	73	7	265	145	All forces irregular.
69	1.5	29	205	1	3	150	175	62	540	420	All forces regular except #2 and #3 - beats.
70	3.2	29	585	1	155	260	340	210	1500	1150	do
71	4.3	29	635	62	99	225	310	250	2000	2050	do
72	0.5	21	58	24	0	50	22	0	105	47	Irregular forces on all. #1, 4, and 5 have regular
73	1.0	21	150	1	27	57	75	0	215	0	forces with beats.
74	2.4	21	335	54	31	170	170	140	665	300	Regular force on #1 and 7 - others irregular.
75	4.8	21	730	90	205	570	490	385	1450	705	Regular forces - beats.
76	0.2	14	44	5	0	4	0	0	88	0	Irregular forces on all.
77	0.9	14	225	12	18	80	75	0	420	0	Irregular forces on all.
78	2.5	14	645	54	55	345	230	200	1150	--	Regular forces on all but #2 and 3 - beats - #2 and 3
79	4.1	14	825	39	92	355	250	405	1350	545	irregular.
80	0.6	6.1	82	27	44	44	22	0	175	90	Appears to have very irregular motion causing very
81	1.2	6.1	170	58	24	95	30	0	270	160	irregular forces on all meters.
82	3.1	6.1	535	200	170	125	150	0	860	510	do
83	4.6	6.1	595	230	180	355	375	110	1100	1400	do

TABLE IV: MOORING FORCES FOR A MODEL ARG-11 IN STERN QUARTERING SEAS

TABLE IV: MOORING FORCES FOR A MODEL ARG-11 IN STERN QUARTERING SEAS

Run No.	Wave Height ft.	Wave Period sec.	1	2	3	4	5	6	7	8	Remarks
84	0.4	99	64	1	1	53	81	10	170	130	Forces are generally regular - some multiple peaks.
85	0.7	102	200	0	22	63	115	11	210	88	do
86	1.1	105	210	0	0	115	170	76	210	88	do
87	1.6	106	250	20	55	200	185	145	270	200	do
88	0.5	82	11	10	0	44	33	0	98	41	Irregular force record.
89	1.0	82	26	19	16	76	56	0	99	87	Regular forces - some multiple peaks.
90	2.6	82	100	44	52	170	115	26	235	185	do
91	4.7	83	140	66	88	180	115	100	270	240	do
92	0.7	48	110	44	51	130	140	24	295	200	Regular forces.
93	1.4	51	165	0	31	140	135	110	505	420	Forces regular - multiple peaks on some meters.
94	3.8	48	275	27	0	355	210	350	675	575	do
95	4.5	48	255	38	11	300	190	395	680	795	do
96	0.5	29	97	8	16	50	48	0	240	77	Irregular forces, all records regular except #2 and
97	1.7	29	265	45	210	275	265	90	595	280	#6.
98	3.3	29	465	0	395	350	235	300	1350	775	Regular forces - multiple peaks - beats.
99	4.3	29	510	47	40	535	230	375	645	2050	Regular forces - multiple peaks - beats.
100	1.1	20	86	0	0	61	45	9	82	120	Irregular forces.
101	1.6	20	270	76	57	160	185	16	475	--	Irregular forces.
102	3.1	20	320	185	310	365	370	0	1100	--	Regular forces - beats - some multiple peaks.
103	4.8	20	385	205	500	625	530	540	1500	1050	Regular forces - beats - some multiple peaks.
104	1.0	13	95	27	27	39	11	0	180	0	Irregular forces - period of force waves longer
105	1.7	13	265	66	130	200	130	35	410	200	than wave period.
106	2.9	13	625	290	94	420	415	495	1350	1050	do
107	4.7	13	660	345	97	435	475	0	1850	--	Regular forces - beats.
108	--	--	No Good								
109	1.7	6.3	0	0	0	0	0	0	37	47	Compression forces only - irregular.
110	5.2	6.4	300	3	0	0	135	0	290	350	Irregular forces on all.
111	3.7	6.5	59	0	0	0	0	0	170	52	Irregular forces on all.

TABLE V: MOORING FORCES FOR A MODEL ARG-11 IN STERN SEAS

Run No.	Wave Height ft.	Wave Period sec.	1	2	3	4	5	6	7	8	Remarks
112	1.0	91	17	32	42	53	80	0	70	67	Regular forces on all tension meters - compression
113	1.4	90	45	19	120	63	120	0	37	25	forces irregular - multiple peaks on some.
114	2.0	89	97	105	98	115	130	0	160	115	do
115	3.0	92	115	145	260	52	180	0	345	122	do
116	0.8	75	48	23	41	99	56	0	150	99	Regular forces on all meters - multiple peaks.
117	1.7	75	130	42	61	175	96	0	135	92	do
118	3.4	75	96	86	165	215	195	0	150	335	do
119	5.8	76	155	88	57	185	290	0	88	480	do
120	0.7	47	78	50	115	75	145	0	200	85	Regular forces on all meters - multiple peaks - beats.
121	1.2	47	105	0	160	77	99	0	235	130	do
122	1.8	47	77	0	230	73	220	6	17	180	do
123	3.5	47	235	14	210	195	215	16	575	470	do
124	0.5	28	105	65	90	64	90	0	145	105	Regular forces - beats - some multiple peaks on
125	1.5	28	230	120	345	275	370	0	435	320	higher waves.
126	4.0	28	480	165	465	445	545	0	725	795	do
127	5.5	28	385	210	430	510	450	0	675	995	do
128	1.0	20	28	17	2	99	67	0	71	100	Irregular forces.
129	1.9	20	220	135	57	185	170	0	475	375	Regular forces - beats.
130	3.5	20	375	235	255	280	365	0	735	390	Regular forces - beats.
131	6.1	20	500	215	410	470	570	0	800	370	Regular forces - beats.
132	0.8	13	58	0	39	51	45	0	110	115	Irregular forces - some beating.
133	2.3	13	200	115	110	145	100	0	270	--	Irregular forces - some beating.
134	4.1	13	460	175	300	465	335	265	880	760	Irregular forces - some beating.
135	5.3	14	550	265	105	305	265	0	1350	0	Regular forces - beats.
136	0.8	6.4	0	0	30	0	10	0	17	0	Very irregular forces on all meters.
137	2.4	6.4	52	0	0	0	24	0	47	0	Very irregular forces on all meters.
138	4.5	6.4	160	31	0	0	76	0	175	85	Very irregular forces on all meters.

Journal of the

WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION

WW 2

J.
With
our st
equat
Howe
—due

which
It is
cordi
It
ment
it ca
lowin
".
er
(F

whic
that
mea
"wei

Fro
misc
line
A
prio
ship

a. P
1. E
D
2. E
th

FORCES INDUCED ON A LARGE VESSEL BY SURGE[a]

Closure by J. T. O'Brien and D. I. Kuchenreuther

J. T. O'BRIEN,[1] A.M. ASCE and D. I. KUCHENREUTHER,[2] J.M. ASCE.—With regard to equation (10) in Dr. Wilson's discussion—which summarizes our statement in paragraph one on page 1571-2 of our paper—we had quoted equation (41) of his paper No. 2460 (Trans. A.S.C.E. 1951, v. 116, pp. 1144). However, we are informed by letter from Wilson that this equation is in error—due to a misprint—and that it should read:

$$R_{max} = 4 \pi W A / T_n (g d)^{\frac{1}{2}} \tag{35}$$

which result can be derived from equations (34a) and (25) of paper No. 2460. It is suggested that copies of Vol. 116 of the Transactions be corrected accordingly.

It seems necessary to present Deacon, Russell and Palmer's (1957) statement on page 78 of their paper, referred to by Wilson and the authors, before it can be discussed further. This paper will be referred to as "Russell" following Wilson. As published it reads. . . .

". . . the expression for the maximum total force in all the cables together emerges (from Russell's analysis of Wilson's paper No. 2460) finally as

$$(R_{max} =) \frac{(\text{mass of the ship})\ (\text{height of the standing wave})}{\text{Wave period}\quad (g \quad \times \text{ water depth})^{1/2}} \ldots"$$

which, when stated symbolically in terms of this discussion and assuming that by "mass of ship" Russell—perhaps more as a physicist than engineer—meant what is usually termed, at least in U. S. A. engineering circles, as "weight," can be written as:

$$R_{max} = 2 W A / T_n (g d)^{\frac{1}{2}} \tag{36}$$

From a comparison of equations (35) and (36) it can be concluded that Russell misquoted Wilson by a factor of $1/2\pi$ and not $g / 2\pi$ as stated by Wilson in line number 18 or page 1884-9 of his discussion.

As to the important controversy—that which engages Wilson beginning just prior to equation (11)—concerning the hydrodynamic behavior of a moored ship driven by a standing wave acting at its node, the authors—at least in the

a. Proc. Paper 1571, March, 1958, by J. T. O'Brien and D. I. Kuchenreuther.

1. Engr., U. S. Naval Civil Engineering Laboratory, Bureau of Yards and Docks, Dept. of the Navy, Port Hueneme, Calif.
2. Engr., U. S. Naval Civ. Eng. Lab., Bureau of Yards and Docks, Dept. of the Navy, Port Hueneme, Calif.

paper under discussion—seem to subscribe to the position taken by Russell (1957) rather than Wilson where the former on page 79 of his paper as cited assumes that:

> ". . . the inertia force is the product of (acceleration of the water particles) and (the mass of the ship and its virtual mass). Accordingly the acceleration of the ship is that of the water particles. . . ."

The authors position may be pointed up by considering first that the expression from Lamb as given by Wilson:

$$m \ddot{x} = F - m'' \ddot{x} \tag{11}$$

can, by substitution of terms as given by Wilson and the authors ($m'' = m' - m = C_M m - m$) be written as:

$$m C_M \ddot{x} = F \tag{37}$$

Now, by combining equation (2), (3), (5) and (6b) of the authors paper we have:

$$m C_M \ddot{x} + C_d A \rho (\dot{x})^2/2 + \sum k_i x^{n_i} = (mg 2 \pi A/L) \sin 2 \pi t/T \tag{38}$$

If damping is considered negligible and there is no restraint to the motion and further that the term on the right hand side of equation (38)—the excitation—is denoted by the symbol F we can write

$$m C_M \ddot{x} = F \tag{39}$$

which is obviously the same as equation (37).

Thus, it would seem that the authors—and perhaps Russell—consider the oscillation of the unmoored and undamped ship to be analogous to that given by Lamb in equation (11) for the motion of a cylinder accelerated by a force through an ideal frictionless fluid, initially at rest.

After this somewhat summary scientific pronouncement, we turn to the comparison of the equation of motion of the authors with that given by Wilson, equation (19). To simplify and make the symbolism of the authors consistent with Wilson we make the following substitutions in the author's equation (38):

$$C_d A \rho (\dot{x})^2/2 = 0 ,$$

$\sum k_i x^{n_i} = C x^n$ where C is an empirically derived constant expressing the elongation behavior of the mooring lines,

$$L = T c = T (g d)^{\frac{1}{2}} ,$$

such that equation (38) can now be written as

$$m C_M \ddot{x} + 0 + C x^n = (m 2 \pi g A/ T (g d)^{\frac{1}{2}}) \sin 2 \pi t/T$$

$$= (m A 2 \pi /T) (g/d)^{\frac{1}{2}} \sin 2 \pi t/T$$

Now we multiply the above equation by $1/m C_M$ and substitute:

$$V = A\,(g/d)^{\frac{1}{2}}$$

$$\sigma = 2\,\pi\,/\,T$$

$$\omega^2 = C/m\,C_M$$

to obtain

$$\ddot{x} + \omega^2 x^n = \left(\frac{1}{C_M}\right) V\,\sigma \sin \sigma t \tag{40}$$

this is compared with the Wilson equation (19) with damping zero:

$$\ddot{x} + \omega^2 x^n = V\,\sigma \sin \sigma t \tag{41}$$

Thus it is apparent that when damping is considered negligible, the author's expression and that of Wilson differ only in that the author's expression for the excitation is less than that of Wilson by factor of $1/C_M$ where, of course, the two become equal when C_M is equal to 1. Therefore, generally any restoring force ($\omega^2 x^n$) calculated from the author's equation (40) would be less than that calculated by use of Wilson's equation (41). In this regard, the authors are unable to add anything significant to Wilson's explanation as to why their calculated values of the restoring force in Figure 13, are greater than those measured—we may have used a mass which is too large—but would suggest that apparently experimental and theoretical deficiencies in explaining Nature are still rampant.

Further, either equation (40) or (41) appear to be identical in principal with those obtained for the approximate mechanical behavior of a mass forced to vibrate without damping against the restrain of a non-linear spring as outlined, for example, by Den Hartog (1947). In the solution of such an equation it is assumed that the motion is sinusoidal and has the "forced" frequency where Den Hartog (1947) states on page 431 with regard to this assumption,

> "this is obviously not true, and the degree of approximation can be estimated only by the seriousness of the deviation from this assumption---"

If sinusoidal motion assumed— $x = x_o \sin \sigma t$ where x_o = maximum amplitude of oscillation—then following Den Hartog, the inertia term in equation (40) can be written as

$$\ddot{x} = -\,\sigma^2 x_o \sin \sigma t$$

so that equation (40) with $C_M = 1$ can be written as

$$\sigma^2 x_o \sin \sigma t + \omega^2 x_o^{\,n} = V\,\sigma \sin \sigma t$$

or

$$\omega^2 x_o^{\,n} = V\,\sigma \sin \sigma t + \sigma^2 x_o \sin \sigma t \tag{42}$$

Equation (42) can be solved graphically—following Den Hartog—by rectangularly plotting $\omega^2 x_o^n$ vertically and amplitude (x_o) horizontally, then noting

that the left side of equation (42) is the non-linear spring characteristic, while the right side expresses a straight line with the ordinate intercept, $V\sigma \sin \sigma t$, and the slope $\tan^{-1} (\sigma^2 x_o \sin \sigma t)$. Thus, values of x_o for various values of $V\sigma \sin \sigma t$ can be obtained by graphical means. Depending on the value of σ there can be from 1 to 3 values of x_o for a particular value of $V\sigma$.

Toward an interesting example—based on material contained in the authors Technical Memorandum as referenced—the lines of the Norton Sound, have been loosened over those used in the authors basic paper and considered to constitute a symmetrical even though non-linear spring such that the combined restoring force term, $\omega^2 x_o^n$ —with C_M taken as 1, m as 835,000 slugs, C as 5000 pounds per $(\text{foot})^n$ and n as 2.01—can be expressed as:

$$\omega^2 x_o^n = (5000/835{,}000\ (1))x_o^{2.01} = .00598\ x_o^{2.01} \tag{43}$$

If equation (43) is substituted in equation (42) along with value for V corresponding to a depth (d) of 32 ft with g conveniently rounded to 32 ft per second per second and maximum amplitudes of oscillation only considered—$\sin \sigma t = 1$ — we obtain

$$.00598\ x_o^{2.01} = A\sigma + \sigma^2 x_o \tag{44}$$

where of course A, the wave amplitude, must be in feet. Now, by application of the graphical technique as presented by Den Hartog (1947) toward solution of equation (44) we obtain values for the amplitude of the ships undamped oscillation (x_o) as excited by standing waves of particular amplitude (A) and frequency ($\sigma = 2\pi$ / wave period, T) as shown in Table below

Table IV

Point	T (sec)	A (ft)	x_o (ft)		
1	25	0	10.5		-10.5
2	25	0.3	11.6	-1.4	- 9.2
3	25	0.7	12.9		
4	30	0	7.5		-7.5
5	30	0.3	8.6	-1.9	- 5.4
6	30	0.7	9.9		

When the wave excitation—$A\sigma$ —and movement (x_o) have the same sign they are considered to be in-phase (ship apparently moving downhill); a difference in sign denotes an out-of-phase relationship (ship apparently moving uphill). Note in some cases that the same excitation can produce both in-phase and out-of-phase ship movement; point No. 2, for example shows one of the former and two of the latter is possible. When the excitation is zero—still water—we have a case of free vibration as shown, for example, in point No. 1.

When the solutions to equation (44) are extended over those presented in Table I and the results plotted, curves of the type given in Figure 15 are obtained which show strikingly the tendency of the ship movement and restoring force to become infinite when the natural period of vibration (when $A\ \sigma = 0$) is approached. Of course, no such thing can occur due to the "fuze" in the system in the form of the mooring lines which tend to break and thereby ruin what elegance there is in this problem.

Note in Figure 15 that both negative as well as positive displacements are plotted where this rather unconventional presentation is made to emphasize those situations where the oscillation (x_o) is 180° out of phase with the excitation ($A\ \sigma$). Usually this phase relation is considered of slight interest—although at the precise point of phase switching many ships could receive a jolt at a level high enough to rouse even the sleepiest seaman and, even worse, to break ropes—in comparison with the amplitudes; therefore, the negative signs are usually disregarded so a presentation is made entirely in the first quadrant.

The authors have obtained a record of such a shift—correlated with changes in mooring forces—by a Landing Ship Tank (LST) as spread moored in the open Gulf of Mexico. This ship shifted the phase of its pitching motion by 180° as the period of the incident wave changed in a very short time from 4 to 7-1/2 seconds where the point of shift is calculated as about 6 seconds.

As explained in many if not all texts on mechanical vibrations, the system, depending upon its period of excitation will be subject to stable—for example, branch 1, 3, 4 and 5 in Figure 15—and unstable—branch 3 and 4-b-motions. Some damping, however slight, must be present in order to permit the ship to cross from in-phase oscillation—periods greater than free period—to out-of-phase oscillation across the zone of transition. (from 4-a to 2 in Figure 15, for example).

It would appear that the free period of oscillation—line designated $A = 0$ in Figure 15--of the ship-line system is one of the dominant design parameters where care should be exercised toward avoiding period coincidence between this period and that of the excitation. A likely operational period of oscillation which is less than—rather than greater than—the free period of oscillation would seem desirable.

A number of investigators, including Abramson and Wilson (1955), during the last 10 years or so have discussed surge oscillation of a ship moored at the node of a standing wave, although none appear to have stretched the mechanical analogy as far as the authors herein. Other modes are not at all well covered. Among the more recent discussions is that of Wilson (1958) —his Paper No. 2460 represents the pioneer effort in this field—which he prepared for the conference on Berthing and Cargo Handling in Exposed Locations, held at Princeton University in October 1958.

The authors hope that this closure has provided in some measure answers to and amplification of the questions raised by Dr. Wilson in his mature and much appreciated discussion of their paper.

REFERENCES

1. Den Hartog, J. P., Mechanical Vibrations, 3rd Edition, McGraw-Hill Company, Inc., 1947, pp. 431-435.

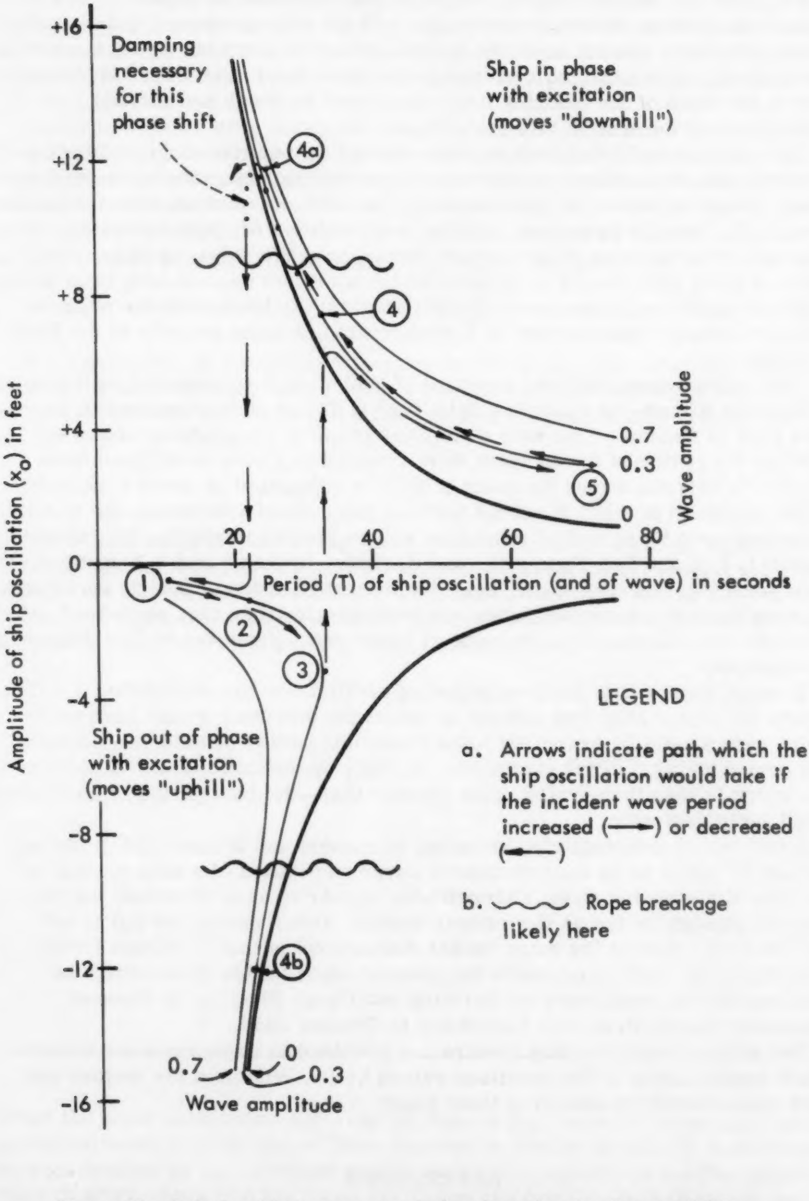


Figure 15. Relationship between period and amplitude of moored ship (and standing wave) oscillation in surge with standing wave amplitude as a parameter.

2. O'Brien, J. T., and Kuchenreuther, D. I., Forces Induced by Waves on the Moored U. S. S. Norton Sound, (A.V.M.-1) Technical Memorandum M-129, U. S. Naval Civil Engineering Laboratory, Port Hueneme, Calif. Apr. 1958.

3. Abramson, H. N., and Wilson, B. W., A Further Analysis of the Longitudinal Response of Moored Vessels to Sea Oscillation, Proc. Joint Mid-West Conf., Solid and Fluid Mechanics, Purdue Univ., Purdue, Sept. 1955.

4. Wilson, B. W., The Energy Problem in the Mooring of Ships Exposed to Waves, vol. on Berthing and Cargo Handling in Exposed Locations. Papers delivered at a meeting of the Princeton Univ. Conference, Oct. 20-21, 1958.

Wl
equat
cient
An ou
avail
Wl
ship,
meth
betw
tribu
big a
for t
have
Tl
illus
Ches
for C
hydr
C
114
Midc
gagi
T
inte
area
usin
runc
T
Den
the
equa
duce
T
Mid
Thi
trav

a.
1.

COLUMBIA BASIN STREAMFLOW ROUTING BY COMPUTER[a]

Discussion by Willard M. Snyder

WILLARD M. SNYDER,[1] A.M. ASCE.—In the description of the routing equations used in the Columbia Basin the author states that routing coefficients used in the computer program were determined by trial and error. An outline of more direct procedure resulting from modifications to an available method(1) may be useful to engineers engaged in routing studies.

When "inflow" and "outflow" data are available, the pattern of the relationship, that is, the time-distribution coefficients can be determined by the method of least squares. There is little difference, in applying this technique, between distribution coefficients which represent a unit hydrograph, or distribution coefficients which serve as routing coefficients through a reach. A big advantage in using the method of least squares is that computer programs for this method are readily available for machines where subroutine libraries have been accumulated.

The method of derivation of distribution coefficients can probably best be illustrated by a numerical example. Figure A-1 shows the hydrographs for Chestuee Creek at Dentville, Tennessee, and, upstream above junction point, for Chestuee Creek at Zion Hill and Middle Creek below Highway 39. These hydrographs show the flood of November 1957.

Chestuee Creek, a tributary of the Hiwasee River, drains a watershed of 114 square miles above Dentville. Drainage areas above the Zion Hill and Middle Creek stations are 37.8 and 32.7 square miles, respectively. These gaging stations are operated by TVA under its Tributary Watershed Program.

The time interval used in plotting Figure A-1 was four hours. This time interval was maintained throughout the analysis. Runoff volumes for the area between the Dentville gage and the two upper gages were estimated, using a rainfall recording gage located in this intermediate area. These runoff volumes are plotted at the top of Figure A-1.

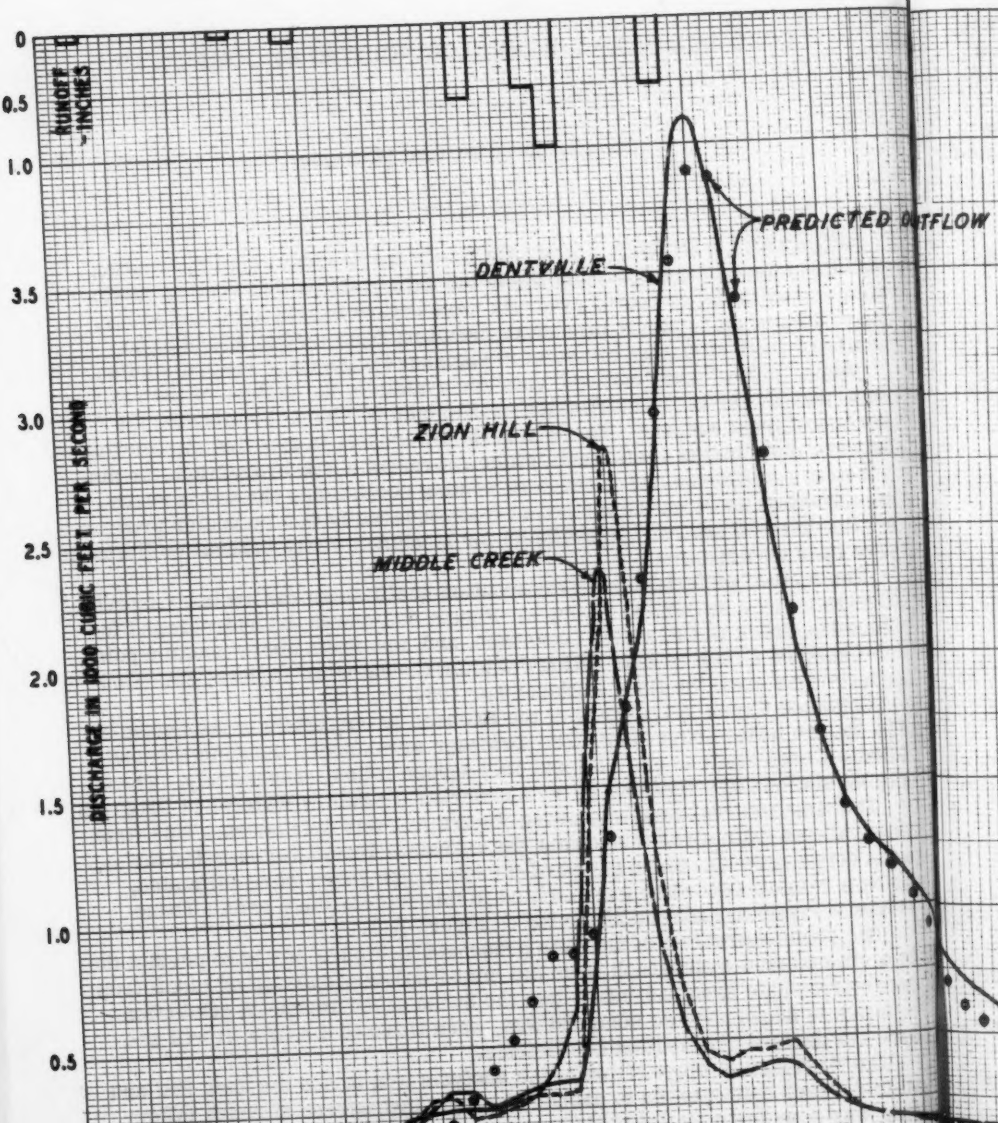
The obvious problem of analysis presented in Figure A-1 is to predict the Dentville outflow hydrograph from the mixed inflow consisting of runoff from the intermediate area and inflow into the heads of the reach. A prediction equation is necessary, so designed that, when fitted to the data, it will produce the desired distribution coefficients.

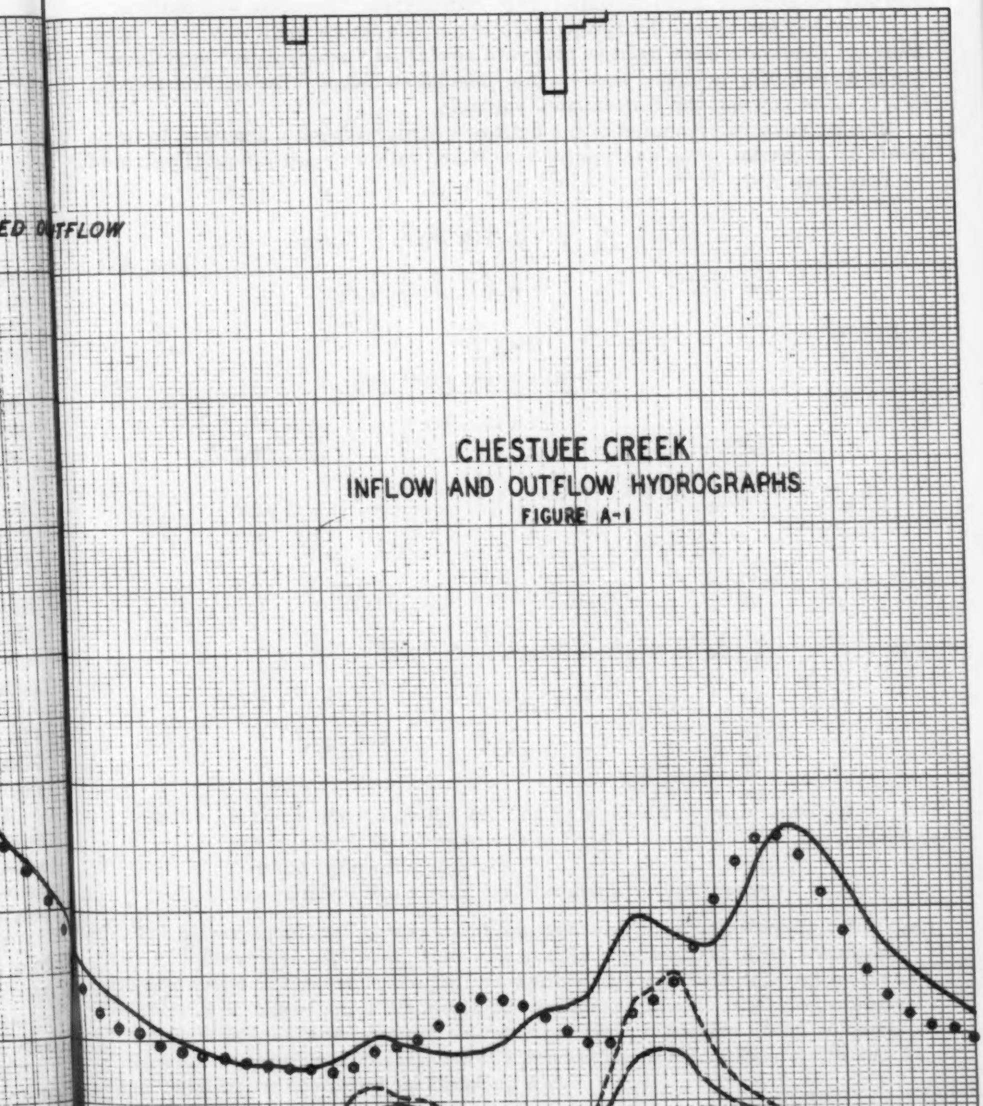
The first step in preparation of the analysis was to add the Zion Hill and Middle Creek hydrographs to produce a single hydrograph of reach inflow. This was done for simplicity and because of approximately equal time of travel. It is not necessary to the solution. The second step in the analysis

---

a. Proc. Paper 1874, December, 1958, by David M. Rockwood.

1. Head, Hydrology Section, Hydraulic Data Branch, Tennessee Valley Authority, Knoxville, Tenn.


0
0.5
1.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
RUNOFF - INCHES
DISCHARGE IN 1000 CUBIC FEET PER SECOND
PREDICTED OUTFLOW
DENTVILLE
ZION HILL
MIDDLE CREEK
14
15
16
17
18
19
20


ED OUTFLOW
CHESTUEE CREEK
INFLOW AND OUTFLOW HYDROGRAPHS
FIGURE A-1
20
21
22
23
24
25
26
27
NOVEMBER 1957

was to devise two numeric functions which are first approximations of the distribution coefficients for the two types of inflow. The choice is not critical and any reasonable approximation from hydrograph inspection will suffice. In Table A-1 are listed the two numeric functions which were assumed. The numeric functions were used to generate so-called "independent" variables for the prediction equation in a manner described below. The outflow hydrograph is, of course, the "dependent" variable.

The successive ordinates of the outflow hydrograph were assumed to be given by computation on successive runoff and inflow quantities as illustrated by Equations A-1

$$\begin{aligned} O_4 = \; & w_1 (5 \, RO_4) \\ & + K_1 (1 \, I_4) \end{aligned} \qquad \text{Eqs. A-1}$$

TABLE A-1

ASSUMED NUMERIC FUNCTIONS

Time	Function for Local Inflow - N	Function for Routing - M
0	0	0
4	5	1
8	10	2
12	8	3
16	6	4
20	5	5
24	5	4
28	4	3
32	4	2
36	3	1
40	3	0
44	3	
48	3	
52	2	
56	2	
60	2	
64	2	
68	2	
72	2	
76	1	
- - -	- - -	
- - -	- - -	
144	1	
148	0	

$$
\begin{aligned}
O_8 = {} & w_1(5\,RO_8 + 10\,RO_4) \\
& + w_2(5\,RO_4) \\
& + k_1(1\,I_8 + 2\,I_4) \\
& + k_2(1\,I_4)
\end{aligned}
\qquad \text{Eqs. A-1}
$$

$$
\begin{aligned}
O_{12} = {} & w_1(5\,RO_{12} + 10\,RO_8 + 8\,RO_4) \\
& + w_2(5\,RO_8 + 10\,RO_4) \\
& + w_3(5\,RO_4) \\
& + k_1(1\,I_{12} + 2\,I_8 + 3\,I_4) \\
& + k_2(1\,I_8 + 2\,I_4) \\
& + k_3(1\,I_4)
\end{aligned}
\qquad \text{Eqs. A-1 (Cont.)}
$$

$$
\begin{aligned}
O_{16} = {} & w_1(5\,RO_{16} + 10\,RO_{12} + 8\,RO_8 + 6\,RO_4) \\
& + w_2(5\,RO_{12} + 10\,RO_8 + 8\,RO_4) \\
& + w_3(5\,RO_8 + 10\,RO_4) \\
& + k_1(1\,I_{16} + 2\,I_{12} + 3\,I_8 + 4\,I_4) \\
& + k_2(1\,I_{12} + 2\,I_8 + 3\,I_4) \\
& + k_3(1\,I_8 + 2\,I_4)
\end{aligned}
$$

where w and k are weighting factors to be found by least squares
RO is runoff from local areas
I is the inflow hydrograph
The numberical values are the numeric functions.

The above equations appear cumbersome, and in detailed notation for time they are unwieldy. However, they are simply expressions of an assumed condition that ordinates of the outflow hydrograph can be predicted by three weighted and lagged applications of the assumed numeric functions. While three applications were chosen, any number two or greater could be used.

If a summation notation is used the prediction equation for outflow can be expressed more concisely as Equation A-2.

$$
O_T = w_1 \sum_{\substack{i=T \\ j=4}}^{\substack{i=T-144 \\ j=144}} N_j\,RO_i + w_2 \sum_{\substack{i=T-4 \\ j=4}}^{\substack{i=T-148 \\ j=144}} N_j\,RO_i + w_3 \sum_{\substack{i=T-8 \\ j=4}}^{\substack{i=T-152 \\ j=144}} N_j\,RO_i
\qquad \text{Eq. A-2}
$$

$$+ K_1 \sum_{\substack{i=T\\ j=4}}^{\substack{i=T-36\\ j=36}} M_j I_i + K_2 \sum_{\substack{i=T-4\\ j=4}}^{\substack{i=T-40\\ j=36}} M_j I_i + K_3 \sum_{\substack{i=T-8\\ j=4}}^{\substack{i=T-44\\ j=36}} M_j I_i$$

Eq. A-2 (Cont.)

where N is the numeric function for local area runoff
M is the numeric function for inflow
and i and j increase by 4-hour intervals.

The product-summation terms NxRO and MxI can be evaluated numerically for each ordinate, O_T. of the observed outflow hydrograph. If these product-summation terms be labeled X_1, X_2, X_3, and Z_1, Z_2, Z_3, then Equation A-2 becomes simply

$$O_T = w_1 X_1 + w_2 X_2 + w_3 X_3 + k_1 Z_1 + k_2 Z_2 + k_3 Z_3 \qquad \text{Eq. A-3}$$

and the coefficients w and k, which are linear in the equation, can be found by least squares. It should be noted that X_2 is X_1 lagged by one period, X_3 is X_1 lagged by two periods, Z_2 is Z_1 lagged by one period, and Z_3 is Z_1 lagged by two periods.

Table A-2 shows the X's, Z's, and outflow, 0, evaluated for the first few periods for the data shown in Figure A-1. The computations, following the form of Equation A-1 are

$$X_{1-1} = (5 \times .05) = .25$$

$$X_{1-2} = (5 \times 0) + (10 \times .05) = .50$$

$$X_{1-3} = (5 \times 0) + (10 \times 0) + (8 \times .05) = .40$$

- -

$$X_{1-12} = (5 \times 0) + (10 \times .10) + (8 \times 0) + (6 \times 0) + (5 \times .05) \cdots + (3 \times .05) = 1.40$$

- -

$$Z_{1-1} = (1 \times .04) = .04$$

$$Z_{1-2} = (1 \times .09) + (2 \times .04) = .17$$

$$Z_{1-3} = (1 \times .14) + (2 \times .09) + (3 \times .04) = .44$$

- -

$$Z_{1-12} = (1 \times .12) + (2 \times .04) + (3 \times .02) + (4 \times .02) + (5 \times .03) + (4 \times .03) + (3 \times .05) + (2 \times .08) + (1 \times .13) = 1.05$$

- -

TABLE A-2

VARIABLES FOR LEAST-SQUARES FITTING

Date	Hour	Period	RO	I	X_1	X_2	X_3	Z_1	Z_2	Z_3	O
14	8	1	.05	.04	.25			.04			.02
	12	2		.09	.50	.25		.17	.04		.09
	4	3		.14	.40	.50	.25	.44	.17	.04	.10
	8	4		.13	.30	.40	.50	.84	.44	.17	.10
	12	5		.08	.25	.30	.40	1.32	.84	.44	.11
15	4	6		.05	.25	.25	.30	1.77	1.32	.84	.12
	8	7		.03	.20	.25	.25	2.07	1.77	1.32	.13
	12	8	.05	.03	.45	.20	.25	2.12	2.07	1.77	.14
	4	9		.02	.65	.45	.20	1.93	2.12	2.07	.13
	8	10		.02	.55	.65	.45	1.47	1.93	2.12	.11
	12	11	.10	.04	.95	.55	.65	1.25	1.47	1.93	.11
16	4	12		.12	1.40	.95	.55	1.05	1.25	1.47	.17
	8	13		.26	1.15	1.40	.95	1.19	1.05	1.25	.18
	12	14		.39	.90	1.15	1.40	1.81	1.19	1.05	.22
	4	15		.55	.80	.90	1.15	3.08	1.81	1.19	.24
	8	16		.63	.75	.80	.90	4.83	3.08	1.81	.26
	12	17		.55	.65	.75	.80	6.98	4.83	3.08	.26

Application of least squares to the complete set of data gave the following values for the weighting coefficients in Equations A-3:

$$w_1 = -.007599$$

$$w_2 = .032219$$

$$w_3 = -.000890$$

$$k_1 = .005696$$

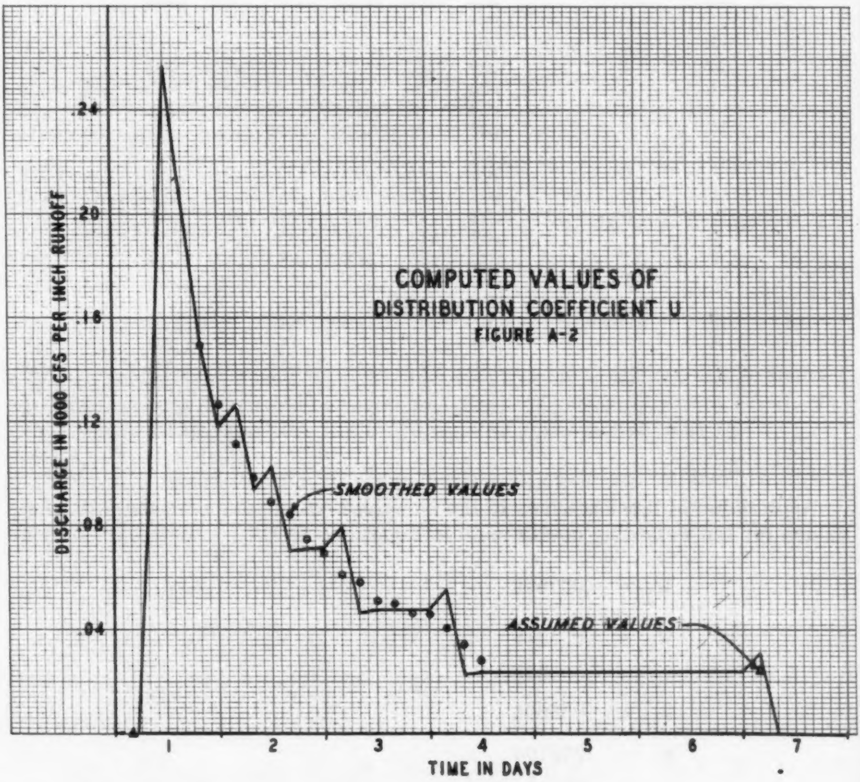
$$k_2 = .049708$$

$$k_3 = -.001069$$

When the weighting coefficients, w, are multiplied by the numeric function, N, and the three resultant functions are added with proper time lag there results a single set of distribution coefficients which is a better approximation to the unit hydrograph than the original function N. The computation is:

$$\begin{aligned} U_1 &= 5\,w_1 \\ U_2 &= 10\,w_1 + 5\,w_2 \\ U_3 &= 8\,w_1 + 10\,w_2 + 5\,w_3 \\ U_4 &= 6\,w_1 + 8\,w_2 + 10\,w_3 \\ U_5 &= 5\,w_1 + 6\,w_2 + 8\,w_3 \\ &- - - - - - - - - - \end{aligned} \qquad \text{Eqs. A-4}$$

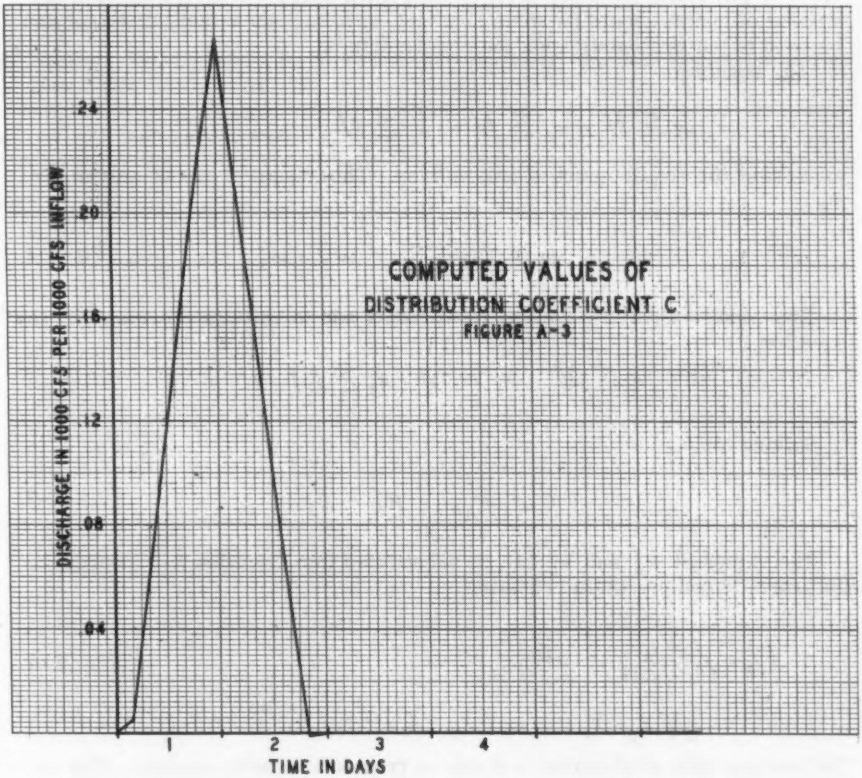
The hydrograph defined by U is plotted in Figure A-2. The approximate unit hydrograph as first derived is not smooth, due principally to the discontinuous nature of the original numeric function. To correct this a smoothing function(2) was applied and the smooth values are also plotted in Figure A-2.



If the weighting coefficients, k, are combined with the numeric function M, in the same manner as w was combined with N. a single set of distribution coefficients result which are the routing coefficients through the reach:

$$
\begin{aligned}
C_1 &= 1k_1 \\
C_2 &= 2k_1 + 1k_2 \\
C_3 &= 3k_1 + 2k_2 + 1k_3 \\
C_4 &= 4k_1 + 3k_2 + 2k_3 \\
C_5 &= 5k_1 + 4k_2 + 3k_3 \\
&- - - - - - - - -
\end{aligned}
\qquad \text{Eqs. A-5}
$$

The routing coefficients, C, are plotted in Figure A-3.



After determination of the coefficients U and C they were applied to the values of runoff, RO, and inflow, I, as given in Table A-2. The resulting predicted outflow hydrograph for Dentville is plotted in Figure A-1. Agreement between the observed and computed outflow is good for the major rise on November 18 and 19, but too rapid an increase is predicted for the smaller rises on November 24 and 26.

Several different procedures, modifications of the above basic technique, are possible. Equation A-2 could be extended to include numeric functions of preceding outflow values. Alternatively, data could be selected from several storms but limited to fixed ranges of outflow. Sets of coefficients U and C could be derived by separate least-squares fitting to each data set.

If more precise values of the coefficients U and C are required, some iterative procedure can be followed. In the numeric example presented, the routed hydrograph for Middle Creek plus Zion Hill could be computed after setting the sum of the coefficients C equal to unity by proportionate adjustment. The routed hydrograph of inflow could be subtracted from the Dentville

outflow. The residual hydrograph of local inflow could be used to adjust the runoff values for each time period by iterative least-squares.(1)

Still another modification of the basic technique is possible that may be of value to particular investigators. Since, for a correct unit hydrograph, the volume of runoff should be some constant, say one, area-inch, this fact can be used to put restraints upon the coefficients w.

The equation

$$w_1 \Sigma N + w_2 \Sigma N + w_3 \Sigma N = \frac{1}{\text{Area Factor}}$$

expresses the requirement of unit volume. This equation can be solved for w_1

$$w_1 = \frac{1}{(A.F.)\Sigma N} - w_2 - w_3 \qquad \text{Eq. A-6}$$

The condition that the routing coefficients add to unity is expressed by the equation

$$k_1 \Sigma M + k_2 \Sigma M + k_3 \Sigma M = 1$$

Solving for k_1

$$k_1 = \frac{1}{\Sigma M} - k_2 - k_3 \qquad \text{Eq. A-7}$$

The values of w_1 and k_1 can be substituted into Equation A-3 to produce the transformed prediction equation

$$O_T - \frac{X_1}{(A.F.)\Sigma N} - \frac{Z_1}{\Sigma M} = \qquad \text{Eq. A-8}$$

$$w_2 (X_2 - X_1) + w_3 (X_3 - X_1) + k_2 (Z_2 - Z_1) + k_3 (Z_3 - Z_1)$$

The lefthand side of Equation A-8 can be treated as a new variable. The coefficients w_2, w_3, k_2, and k_3 can be found by least squares, using the new dependent variable and the X - differences and Z - differences as new independent variables. Coefficients w_1 and k_1 can be found from Equations A-6 and A-7 after the other coefficients are known. It should be pointed out that imposing the restraints of Equations A-6 and A-7 has reduced the degrees of freedom of the prediction equation from six in Equation A-3 to four in Equation A-8, and some less of fitting precision will result.

The discussion presented here suggests somewhat lengthy numerical procedures. But these computations are basically repetitious and not complex. Many or all of the steps can be programmed for electronic computation. Since the computations in many instances are reducible to matrix or vector operations, subroutines for much of the work would be available for many computers.

REFERENCES

1. Snyder, Willard M. "Hydrograph Analysis by the Method of Least Squares" Proceedings of the American Society of Civil Engineers, Volume 81, Paper No. 793, September 1955.

2. Nielsen, Kaj L. "Methods in Numerical Analysis" The MacMillan Company, New York, 1957, p. 291.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY 7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

VOLUME 84 (1958)

JUNE: 1658(AT1), 1659(AT1), 1660(HY3), 1661(HY3), 1662(HY3), 1663(HY3), 1664(HY3), 1665(SA3), 1666 (PL2), 1667(PL2), 1668(PL2), 1669(AT1), 1670(PO3), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675 (PO3), 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), 1681(SA3), 1682(SA3), 1683(PO3), 1684 (HY3), 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)[c], 1689(PO3)[c], 1690(HY3)[c], 1691(PL2)[c].

JULY: 1692(EM3), 1693(EM3), 1694(ST4), 1695(ST4), 1696(ST4), 1697(SU2), 1698(SU2), 1699(SU2), 1700(SU2), 1701(SA4), 1702(SA4), 1703(SA4), 1704(SA4), 1705(SA4), 1706(EM3), 1707(ST4), 1708(ST4), 1709(ST4), 1710 (ST4), 1711(ST4), 1712(ST4), 1713(SU2), 1714(SA4), 1715(SA4), 1716(SU2), 1717(SA4), 1718(EM3), 1719 (EM3), 1720(SU2), 1721(ST4)[c], 1722(ST4), 1723(ST4), 1724(EM3)[c].

AUGUST: 1725(HY4), 1726(HY4), 1727(SM3), 1728(SM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733 (PO4), 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742 (PO4), 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758 (ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767 (WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775(IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784(IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

OCTOBER: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), 1798(HW3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806(HY5), 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815(ST6), 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(EM4), 1823(PO5), 1824(SM4), 1825(SM4), 1826(SM4), 1827(ST6)[c], 1828(SM4)[c], 1829(HW3)[c], 1830(PO5)[c], 1831(EM4)[c], 1832(HY5)[c].

NOVEMBER: 1833(HY6), 1834(HY6), 1835(SA6), 1836(ST7), 1837(ST7), 1838(ST7), 1839(ST7), 1840(ST7), 1841(ST7), 1842(SU3), 1843(SU3), 1844(SU3), 1845(SU3), 1846(SU3), 1847(SA6), 1848(SA6), 1849(SA6), 1850 (SA6), 1851(SA6), 1852(SA6), 1853(SA6), 1854(ST7), 1855(SA6)[c], 1856(HY6)[c], 1857(ST7)[c], 1858(SU3)[c].

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867 (ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876 (WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

VOLUME 85 (1959)

JANUARY: 1892(AT1), 1893(AT1), 1894(EM1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), 1900(HW1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), 1909(ST1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(AT1)[c], 1917(EM1)[c], 1918(HW1)[c], 1919(HY1)[c], 1920(PL1)[c], 1921(SA1)[c], 1922(ST1)[c], 1923(EM1), 1924(HW1), 1925(HW1), 1926 (PL1), 1927(HW), 1928(HW1), 1929(SA1), 1930(SA1), 1931(SA1), 1932(SA1).

FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO), 1957(CO1), 1958(CO1), 1959(CO1).

MARCH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968(SA2), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977(WW1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)[c], 1986 (IR1)[c], 1987(WW1)[c], 1988(ST3)[c], 1989(HY3)[c].

APRIL: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998 (SM2), 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007 (HW2)[c], 2008(EM2)[c], 2009(ST4)[c], 2010(SM2)[c], 2011(SM2)[c], 2012(HY4)[c], 2013(PO2)[c].

MAY: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)[c], 2041(PL2)[c], 2042(PP1)[c], 2043(ST5)[c], 2044(SA3)[c], 2045(HY5)[c], 2046(PP1), 2047(PP1).

JUNE: 2048(CP1), 2049(CP1), 2050(CP1), 2051(CP1), 2052(CP1), 2053(CP1), 2054(CP1), 2055(CP1), 2056 (HY6), 2057(HY6), 2058(HY6), 2059(IR2), 2060(IR2), 2061(PO3), 2062(SM3), 2063(SM3), 2064(SM3), 2065 (ST6), 2066(WW2), 2067(WW2), 2068(WW2), 2069(WW2), 2070(WW2), 2071(WW2), 2072(CP1)[c], 2073(SM3)[c], 2074(PO3)[c], 2075(ST6)[c], 2076(HY6)[c], 2077(SM3)[c], 2078(WW2).

c. Discussion of several papers, grouped by divisions.

AMERICAN SOCIETY OF CIVIL ENGINEERS

OFFICERS FOR 1959

PRESIDENT
FRANCIS S. FRIEL

VICE-PRESIDENTS

Term expires October, 1959:
WALDO G. BOWMAN
SAMUEL B. MORRIS

Term expires October, 1960:
PAUL L. HOLLAND
LLOYD D. KNAPP

DIRECTORS

Term expires October, 1959:
CLINTON D. HANOVER, Jr.
E. LELAND DURKEE
HOWARD F. PECKWORTH
FINLEY B. LAVERTY
WILLIAM J. HEDLEY
RANDLE B. ALEXANDER

Term expires October, 1960:
PHILIP C. RUTLEDGE
WESTON S. EVANS
TILTON E. SHELBURNE
CRAIG P. HAZELET
DONALD H. MATTERN
JOHN E. RINNE

Term expires October, 1961:
THOMAS J. FRATAR
EARL F. O'BRIEN
DANIEL B. VENTRES
CHARLES W. BRITZIUS
WAYNE G. O'HARRA
FRED H. RHODES, JR.
N. T. VEATCH

PAST PRESIDENTS
Members of the Board

MASON G. LOCKWOOD
LOUIS R. HOWSON

EXECUTIVE SECRETARY
WILLIAM H. WISELY

ASSISTANT SECRETARY
E. LAWRENCE CHANDLER

ASSISTANT TREASURER
ENOCH R. NEEDLES

PROCEEDINGS OF THE SOCIETY

HAROLD T. LARSEN
Manager of Technical Publications

PAUL A. PARISI
Editor of Technical Publications

MARVIN SCHECHTER
Assistant Editor of Technical Publications

COMMITTEE ON PUBLICATIONS

HOWARD F. PECKWORTH, *Chairman*

PHILIP C. RUTLEDGE, *Vice-Chairman*

E. LELAND DURKEE
TILTON E. SHELBURNE
CHARLES W. BRITZIUS
FRED H. RHODES, JR.

WW2

JUNE 1959 — 25
VOLUME 85

NO. WW 2
PART 2

Your attention is invited

NEWS OF THE WATERWAYS AND HARBORS DIVISION OF ASCE

AMERICAN SOCIETY OF CIVIL ENGINEERS
FOUNDED 1852

JOURNAL OF THE WATERWAYS AND HARBORS DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

June, 1959

Revised Statements of Purpose for Division

At the January meeting of the Water Resources Coordinating Committee a recommendation was made that each division study its own organization and furnish specific recommendations to the Coordinating Committee with respect to changes in the purpose of the division and its committees. Insofar as the Waterways and Harbors Division is concerned, the Coordinating Committee recommended more definite expression of the Divisions interests with respect to floods, dams, and stream channels to avoid conflict with other divisions having related interests. In addition, the Coordinating Committee recommended amplification of the purposes of certain technical committees to include the effect of droughts, conservation and reuse of water, and abatement of industrial wastes and sedimentation.

To implement the recommendations of the Coordinating Committee, the Waterways and Harbors Division Executive Committee at its March meeting rephrased the statement of purpose of the Division and several technical committees. These revised statements of purpose are as follows:

"WATERWAYS AND HARBORS DIVISION
(Authorized as Waterways Division June 16, 1924; Harbors added February 13, 1956)

Purpose: The advancement and dissemination of engineering knowledge concerning the utilization of waterways and harbors, and the protection and development of ocean and other water frontage. This includes studies in the following fields: floods and methods of protection, droughts and methods of alleviation, tidal action, wave action, and unusual phenomena which affect the stability of shore lines and the navigable capacity or usefulness of waterways and harbors; port development and operation; types of structures and equipment used in navigation, shore protection and flood control engineering, their design, construction, maintenance, and the suitability of materials for such use; and the economics of waterborne transportation.

Note: No. 1959-25 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, WW 2, June, 1959.

COMMITTEE ON NAVIGATION AND FLOOD CONTROL FACILITIES

Purpose: To study and report on problems of planning, design, construction, operation and maintenance of locks, dams, levees, walls and channels to meet the requirements of navigation and flood control on intercoastal and inland waterways with particular attention to types and dimensions of locks and dams, reuse of water by recirculation, types of gates, valves, culverts and other appurtenances, and investigation of practices and trends in barge, towboat and ship sizes and in operating practices.

COMMITTEE ON THE REGULATION AND STABILIZATION OF RIVERS

Purpose: To study and report on the problems of planning, design, and maintenance of navigable channels by means of dredging, abatement of industrial wastes, sedimentation control, channel alinement, permeable dikes, training dikes, groins, bank stabilization works, and flow regulation, together with effects and methods of protection from floods and droughts.

COMMITTEE ON COASTAL ENGINEERING

Purpose: To study and report on methods of stabilizing and improving coastal inlets and shores which are exposed to the action of waves and currents, including hurricane effects, and on the design, construction and maintenance of engineering works for these purposes."

Coordinating Committee on Transportation

In January 1959 the Task Committee on Transportation submitted its report to the Board of Direction of ASCE. The Task Committee included Mr. Roger H. Gilman, member of the Waterways and Harbors Division Executive Committee. The Task Committee reported in part as follows:

"In each form of transportation engineers naturally consider and measure existing and future needs, particularly its relation to existing and potential compacities. It is inevitable also that planning in one form of transportation requires consideration of how it affects and is affected by other forms of transportation. Furthermore, since transportation is assuming increasing importance in our economy, and indeed has become one of the principal indexes of economic health and growth, it is necessary that engineers consider the relationship of each form of transportation, not only with respect to all other forms but with respect to the very life of the communities, the States and the Nation.

While preliminary planning for each form of transportation can be done fairly effectively, there is a wide area of conjecture in determining future needs. This is unfortunate because transportation facilities by their nature are high in cost and long-lasting, and prognostications wide of the mark can be costly and wasteful. The margin of error is steadily being narrowed as research continues and measurements are made of trends. It may well be, however, that this margin of error can be narrowed still further if a national transportation policy is developed which could be used as a reasonable guide toward probable future requirements.

The importance of transportation in its various forms in the national economy can be gauged by its relation economically to the gross national product. In 1956 when the gross national product was $419-billion, the total expenditure for passenger transportation was $33.8-billion, and for freight transportation $32-billion, a total of $65.8-billion or 15.7 per cent of the total. The projections to 1980 envision a gross national product of $880-billion, of which passenger transportation will cost $85-billion, freight transportation $80-billion, a total of $165-billion, or 18.8 per cent of the total, all in terms of 1956 dollars. The increasing importance of transportation in our national economy is evident in these figures.

Since transportation in its various forms is, and will continue to be, such an important element in our economy it will demand the attention of engineers to an increasing degree. Regardless of whether or not a national transportation policy is formulated, engineers will be called upon not only to design and construct the transportation plant but to do the preliminary planning therefor. They will have to be fully conversant with, indeed expert in, the planning, development and operation of the transportation system.

To insure that the objectives of the Society represented by its technical divisions; investigation, research, dissemination of information, etc., are met with respect to transportation, your Task Committee on Transportation has examined the organization of the technical divisions. It was concluded that major changes in the divisions or their administrative committee are not necessary but that an organization form is needed to insure cooperation between the several divisions interested in transportation and between these divisions and other organizations not under Society sponsorship. The committee further concluded that certain voids in the divisional organization regarding transportation should be filled and perhaps certain overlaps or duplications eliminated.

The Committee feels that the Society should have a Coordinating Committee on Transportation reporting directly to the Committee on Division Activities. This committee would be advisory to and cooperate with the following divisions:

Air Transport
City Planning
Highway
Pipeline
Power
Waterways and Harbors.

The functions of the Coordinating Committee would be fourfold as follows:

1. Encourage more exchanges and contacts between the several technical divisions interested in transportation.
2. Act as a focal point for initiating studies and disseminating information on broad transportation policies.
3. Make a detailed study of the existing technical divisions and recommend changes to fill voids and possibly eliminate overlaps.

4. Study the relationship of the technical divisions to the work of organizations not directly connected with the Society but which are also interested in broad transportation policies."

The Task Committee suggested many areas of interest which did not appear to be adequately covered by the existing technical divisions of ASCE. For the Waterways and Harbors Division, the Task Committee indicated that the field of water transportation studies could be covered more fully.

The Task Committee, in recommending that a coordinating committee of the Society be set up indicated that many non-society organizations could be contacted by such a coordinating committee. In waterways and harbors matters, the Committee suggested the following non-ASCE organizations:

1. American Association of Port Authorities
2. National Rivers and Harbors Congress
3. American Waterways Operators
4. American Merchant Marine Institute
5. Permanent International Association of Navigation Congresses.

The Board of Direction of ASCE received the report of the Task Committee at the Los Angeles Convention, and immediately proceeded to form a Coordinating Committee on Transportation. Mr. Thomas J. Fratar, a member of the Division Committee on Ports and Harbors was named as Chairman of the Coordinating Committee. The other members of the Committee are presently being appointed.

1960 River and Harbor Engineering Conference at Princeton University

The Department of Civil Engineering at Princeton University, in connection with its program of graduate level education in river and harbor engineering, is planning a conference on shipping and navigation problems of the Great Lakes and St. Lawrence Seaway to be held at Princeton on January 19 and 20, 1960. The conference will be similar in character to the October 1958 conference on Berthing and Cargo Handling in Exposed Locations, which was attended by 82 representatives of 29 corporations, 45 representatives of engineering and contracting firms, 16 engineers from the Governments of the United States and Canada and 29 representatives of educational institutions.

At the present time a series of seven papers is planned for the conference.

1. <u>Surface elevations of the Great Lakes.</u> The problem of maintaining satisfactory minimum elevations.
2. <u>Seiches in the Great Lakes and their Effect on Navigation.</u>
3. <u>Navigation Problems of the St. Lawrence Seaway and the Great Lakes.</u> Special emphasis to be given to the probability of bottlenecks developing in one or more sections of the system.
4. <u>The Ice Problem.</u>
5. <u>The Ocean-Lake Ship.</u> The feasibility of ships that can be profitably operated within the Great Lakes and on ocean routes.
6. <u>Great Lakes Port Developments.</u>
7. <u>The effect of the Seaway on the Great Lakes Traffic Pattern.</u>

Expansion of Technical Activities of Division

At the March meeting of the Division Executive Committee, Mr. Richard Eaton, Chairman of the Committee on Coastal Engineering, submitted a compilation showing the Society Technical Organization for each division. This compilation is as follows:

A.S.C.E. Technical Organization

	Technical Committees		
Division	Regular	Task	Total
Air Transport	4		4
City Planning	6		6
Construction	4	4	8
Engineering Mechanics	7		7
Highway	10		10
Hydraulics	6	15	21
Irrigation and Drainage	6	1	7
Pipeline	6	13	19
Power	4		4
Sanitary	13		13
Soil Mech. and Found.	8	4	12
Structural	9	26	35
Surv. and Mapping	8		8
Waterways and Harbors	5	2	7

The Executive Committee is reviewing this material to determine if the Waterways and Harbors Division is understaffed for its technical objective and if additional Task Committees are needed to work in specific areas.

Technical Sessions on Waterfront Construction

At the Los Angeles Convention, the Committee on Ports and Harbors under the Chairmanship of Ben Nutter discussed plans for future convention programs. The Committee is considering presentation at future conventions of symposiums on construction materials for waterfront structures and on types of wharf construction.

The Committee also discussed formation of subcommittees to investigate the feasibility of preparing a slide film and a manual on ports and harbors. Expansion of the membership of the Committee to broaden its geographical representation was also considered.

In addition to Chairman Nutter, other Committee members who attended the meeting were: Tom Fratar, Bob Hoffmaster, John Luttman-Johnson, and George Treadwell.

Change in Committee Chairmanship

Mr. Richard Eaton has resigned as Chairman of the Committee on Coastal Engineering. Mr. Eaton is Chief Technical Advisor, Beach Erosion Board, Corps of Engineers, 5201 Little Falls Rd., N. W., Washington, D. C.

The Executive Committee has appointed Mr. Thorndike Saville, Jr. as Chairman of the Committee on Coastal Engineering. Mr. Saville is Consulting Water Resources & Coastal Engineer, Director, Science and Technical Center Study, University of Florida, Medical Science Building, Room M-135, Gainesville, Florida.

7th Conference on Coastal Engineering

The 7th Conference on Coastal Engineering will be held at The Hague in 1960. The host for the conference is the Rijkswaterstaat. Mr. J. B. Schijf of this organization was recently in the United States.

The conference will be held on August 22-27, 1960 in the Kurhaus (a convention hall) in the town of Scheveningen. This is a resort and residential community which is part of The Hague.

Attendance Record Set

At the Waterways and Harbors Division program at the Los Angeles Convention a new attendance record for the Division was set, with attendance at the four sessions ranging from 75 to 200. This record indicates the desirability of joint sessions with other divisions since larger attendance can be obtained. The Los Angeles Sessions were presided over by Professor Joe Johnson of the University of California.

Correction

In the March 1959 issue of the Waterways and Harbors Division Newsletter, Mr. J. G. Turney was incorrectly identified as the Local Section Representative for Texas of the Committee on Cooperation with Local Sections. The Texas Representative of this committee is Professor Basil W. Wilson of the Department of Oceanography and Meteorology of the Agricultural and Mechanical College of Texas. Mr. Turney is a member of the Board of Commissioners of the Harris County-Houston Ship Channel Navigation District.

For Your Calendar

October 19-23, 1959	ASCE, Washington, D. C. Convention
January 19-20, 1960	Princeton Conference
March 7-11, 1960	ASCE, New Orleans Convention
June 19-23, 1960	ASCE, Reno Convention
October 9-13, 1960	ASCE, Boston Convention
April 10-15, 1961	ASCE, Phoenix Convention
October 16-20, 1961	ASCE, New York Convention
February, 1962	ASCE, Houston Convention
May, 1962	ASCE, Omaha Convention
October 15-19, 1962	ASCE, Detroit Convention

Newsletter Publication

The next issue of The Waterways and Harbors Division Journal will be in September 1959. The deadline for submission of copy for that issue is July 31, 1959. If you have any material that might be usable in the Newsletter which will accompany the Journal, please send it to:

Austin E. Brant, Jr.
Editor, Waterways and Harbors Division Newsletter
Tippetts-Abbett-McCarthy-Stratton
375 Park Avenue
New York 22, N. Y.

Please note the new address for Newsletter material.